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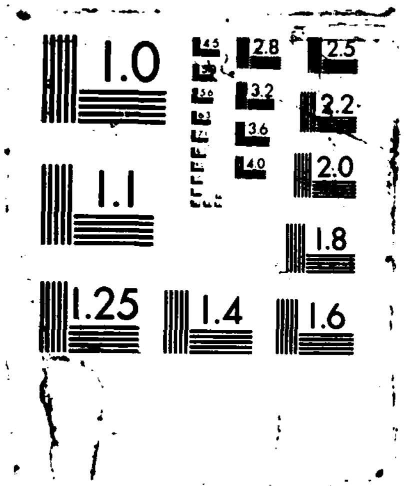
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BEHAVIOR OF A LARGE-SCALE PILE GROUP SUBJECTED TO CYCLIC LATERAL LOADING

AD-A193 498

by

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A large-scale group of nine steel-pipe piles in a closely spaced arrangement was subjected to cyclic lateral loading with water above the ground surface. The piles were 10.75 in. in diameter and 40 ft long and were installed at a center-to-center spacing of 3-pile diameters. The soils at the Houston, Texas site consisted of a stiff, overconsolidated clay. In an attempt to model storm loading of offshore structures, the loading was electronically controlled to follow a sinusoidal curve of deflection versus time with a 30-sec period. All of the piles in the group were extensively instrumented to obtain measurements of shear in the individual piles and bending moment as a function of depth. Results were compared with the results of a similar load test on a single pile that was about 30 ft away.

Several methods of analysis that are currently in use in design were reviewed and the

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predictions that were made using these methods were compared with measured results. The results emphasize the highly nonlinear nature of the pile-soil-pile interaction. A substantial reduction in ultimate soil resistance was measured in the group piles relative to that of a similarly loaded single pile for both the first cycle and for 100 cycles. Interim recommendations are given for the design of closely-spaced-pile groups subjected to lateral loading.

PREFACE

This study was performed by the Geotechnical Engineering Center, Bureau of Engineering Research, The University of Texas at Austin, under contract to the US Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, for the Minerals Management Service, US Department of Interior, the Department of Research, Federal Highway Administration, and the US Army Engineer Division, Lower Mississippi Valley. The study was performed under Contract No. DACW 39-83-C-0061.

This report was prepared by Mr. Dan A. Brown and Dr. Lymon C. Reese, University of Texas at Austin, and reviewed by Mr. Gerald B. Mitchell, Chief, Engineering Group, Soil Mechanics Division (SMD), Geotechnical Laboratory (GL), WES. General supervision was provided by Mr. Clifford L. McAnear, Chief, SMD, and Dr. William F. Marcuson III, Chief, GL.

COL Dwayne G. Lee is Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

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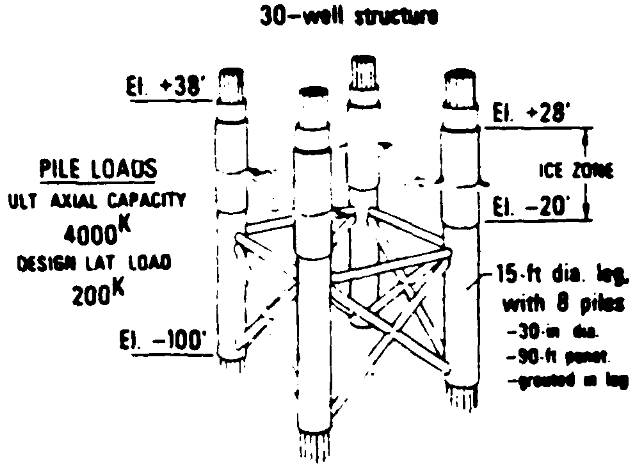
CHAPTER 1

INTRODUCTION

The problem of designing the piles in a closely-spaced group for lateral loading is not new; highway-bridge foundations, lock and dam structures and waterfront structures have utilized groups of piles for support of such loadings. During the last 20 years, offshore development has moved into areas of severe environmental conditions and into deeper water and has prompted increased interest in the behavior of pile groups. Ice loads in the arctic and severe storm loads in the North Sea have encouraged the design of platforms with fewer tower legs to minimize exposure, thus resulting in more concentrated foundation loading. A solution to the problem of supporting these increased loads has been the use of driven piles in closely spaced groups, as illustrated in Fig. 1.1. Piles in such close proximity subjected to lateral loading are influenced by the existence of similarly loaded piles nearby, a situation characterized by Wright (1982) as one of "pile-soil-pile interaction."

The purpose of this study is to investigate the nature of this "pile-soil-pile interaction" and to attempt to identify important factors which influence the behavior of groups of piles in clay subjected to lateral load. An emphasis is placed on developing important principles to be included in the design of pile groups for lateral load; engineers presently have little quantitative information upon which to



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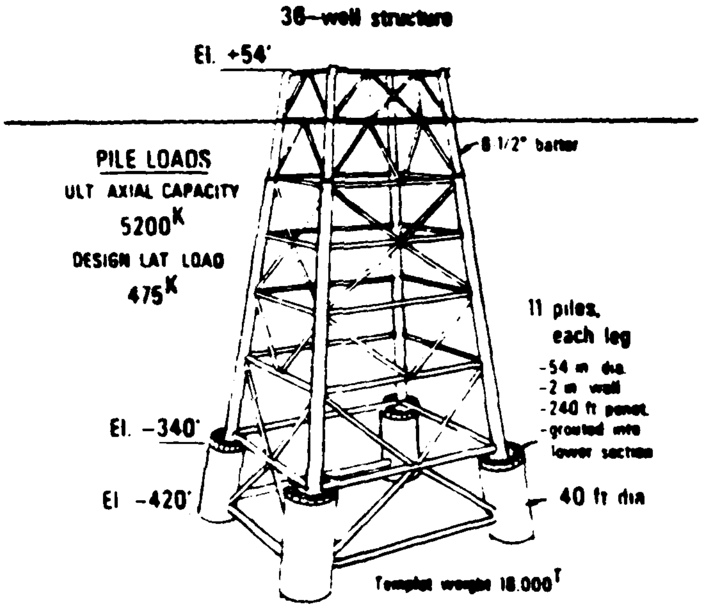


North Sea

Fig. 1.1 Examples of the Use of Closely Spaced Pile Groups, (after McClelland, 1972)

develop judgement. To this end, a major part of this study is devoted to conducting a large scale, well-instrumented experiment designed to provide such quantitative information.

A review of present understanding of the behavior of piles and pile groups is made to provide a background for the experimental study. This review is begun in Chapter 2 with a discussion of common design methodology for single piles in clay subjected to lateral loading. Concepts of load transfer using the Winkler model and p-y curves are reviewed, along with correlations of load transfer to soil properties. The effects of cyclic loading on pile response are also discussed.

Summarized in Chapter 3 are concepts and observations on the effect of pile installation on the behavior of piles and the surrounding soil, an aspect of the pile problem which is not well understood. Commonly used design procedures correlate pile behavior with soil properties that are measured prior to pile installation; the correlations are semi-empirical in that they are based principally on experimental results. An appreciation of the effects of pile installation is crucial to prevent misapplication of such correlations.

Major design considerations for piles in closely-spaced groups are outlined in Chapter 4. These design approaches are contrasted with those used for single piles. Several commonly used analytical models are described, and the fundamental assumptions used in these models are identified.

Chapter 5 contains a review of previous experimental work with pile groups subjected to lateral loading and thus places the current

experimental study within the framework of the history of the subject. The discussion reveals the lack of carefully performed, well instrumented, large-scale tests; the design of pile groups for lateral loading is based upon a very limited data base.

The testing program is described in detail in Chapter 6. This description includes a discussion of the site conditions and soil properties as well as a description of the instrumentation and loading procedures. Observations of the behavior of the piles and surrounding soil during the test are also included.

The measured response of the pile group is presented in Chapter 7. Variation in behavior of the individual piles with respect to geometric position is examined. Results of an earlier single-pile, load test are also presented and compared with the results of the group-load test.

The methods of analysis described in Chapter 4 are used to predict the behavior of the group; these predictions are compared with measured test results in Chapter 8. The single-pile results are used to "calibrate" these analytical models to the extent possible. The suitability of the fundamental concepts used as a basis for the various methods is examined.

As one might expect, the analytical models described in Chapter 8 leave room for improvement. In an attempt to respond to the challenge of "what would <u>you</u> do if you had to design a pile group for cyclic lateral loading," recommendations for design are provided in Chapter 9. As with most experimental investigations, a thorough and quantitative understanding of fundamental principles does not emerge;

rather, some understanding is gained, some established concepts are challenged, and many more questions are raised. The recommendations given in Chapter 9 are offered as guidelines for design until a better understanding of important parameters permits development of more rigorous analytical models.

Chapter 10 contains a brief summary of the results of the study and the conclusions that were made. Recommendations for further research are also given.

CHAPTER 2

BEHAVIOR OF SINGLE PILES UNDER LATERAL LOAD

INTRODUCTION

An understanding of the behavior of pile groups under lateral load first requires a consideration of the behavior of single piles under similar loading conditions. The purpose of this chapter is to provide a very brief review of the current practice in the design of single piles for lateral load as a basis for subsequent discussions of pile group behavior Much research remains to be done before a thorough and complete understanding of single piles is achieved, a fact which makes the pile group problem all the more difficult.

Three broad approaches to the analysis of single piles under lateral loading have received the attention of geotechnical engineers. These are:

1. consider the pile as a beam on an elastic continuum (Poulos, 1971),
2. model the soil as a series of closely spaced discrete springs with no coupling of adjacent soil elements (Winkler, 1867), and
3. model the soil using the finite element approach (Desai and Appel, 1976).

The Winkler assumption is used in the vast majority of design procedures currently in use due to the fact that it is simple to use and is

easily modified to account for nonlinear soil response and loss of soil resistance during cyclic loading. Analytical procedures using this approach have been "calibrated" using experimental results, thus encouraging their use in routine design. Elastic continuum models are used frequently for problems involving only small soil strains, such as dynamically loaded foundations. The finite element method has not realized widespread use for the design of piles under lateral load due to problems in establishing constitutive laws for soil at large strains, as well as the generally greater computational effort required. The finite element approach presently remains a research tool.

The discussion which follows will concentrate on the approach using the Winkler assumption for soil response.

GOVERNING EQUATIONS

The governing differential equation for a beam on elastic (Winkler-type) soil has been derived by Hetenyi (1946) and is presented as

$$EI \frac{d^4y}{dx^4} + P_x \frac{d^2y}{dx^2} - p - W = 0$$

where:

P_x = axial load on the pile,

y = lateral deflection of the pile at a point x along the length of the pile,

p = soil reaction per unit length,

EI = flexural rigidity, and

W = distributed load along the length of the pile.

Other beam formulae which are useful in the analysis are:

$$EI \frac{d^3y}{dx^3} = V$$

$$EI \frac{d^2y}{dx^2} = M$$

$$\frac{dy}{dx} = S$$

where:

V = shear,

M = bending moment of the pile, and

S = slope of the elastic curve.

The solution to the governing differential equation for a given problem is typically obtained using finite difference techniques, and is a well-established procedure in geotechnical engineering. The key to successful use of the method is an accurate description of the soil response to load.

SOIL RESPONSE

General

Soil response in the governing equation above is typically described by relating soil resistance, p, to the deflection, y, at a given point on the pile, using a nonlinear relationship commonly called a p-y curve. Note that the soil resistance, p, is really the integrated change in soil stress on the pile for a given pile increment. The distribution of pressure on the face of the pile is therefore not considered directly. The concept of the p-y curve is well

established in the technical literature. Figure 2.1 illustrates a typical p-y curve along with a representation of the beam solution using the Winkler assumption. Note that the soil resistance is represented by a nonlinear spring coupled to a sliding block; the sliding block is used to represent the soil resistance at which shearing and unlimited soil deflection takes place. The p-y relationship is implemented in the code COM622 (Reese, 1977) by computing a secant modulus of soil reaction, E_s (units of force/area), where:

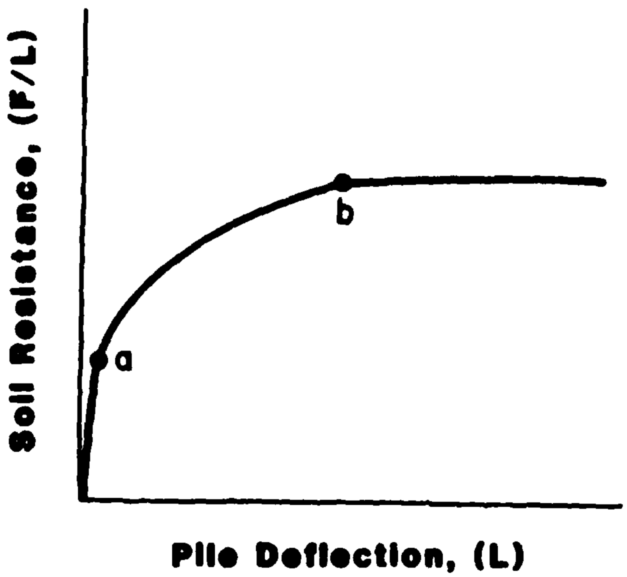
$$-p = E_s y$$

After solution of the series of equations (representing each pile increment and p-y curve), new values of E_s are computed and the solution repeated. This iteration continues until convergence to some small error in the p-y and E_s relationship.

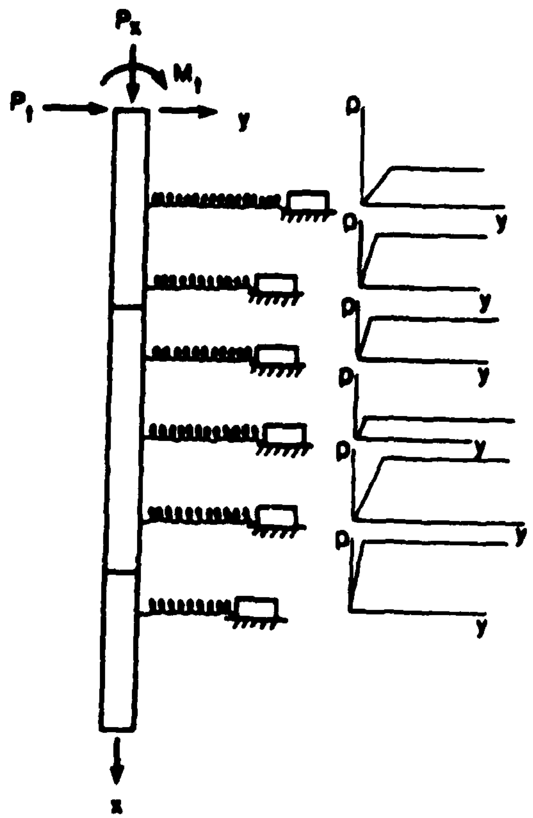
Analytical Bases for p-y Curves in Clay

Although the methods presently used to construct p-y curves were derived primarily from duplicating experimentally determined curves, the methods are based on soil mechanics principles. A brief review of these principles is provided below; with details of existing recommendations for constructing p-y curves presented by Reese (1984).

Initial Modulus. Terzaghi (1955) outlined recommendations for a coefficient of subgrade reaction using the theory of elasticity and some experimental results. He proposed a linear relationship between deflection and the resistance offered by the soil during deflection, which he stipulated to be valid up to no more than one-half of the



Conceptual p-y curve



Model of a pile under lateral loading showing concept of soil response curves

Fig. 2.1 p-y Concept (after Reese, 1984)

maximum bearing stress. For stiff clays, Terzaghi suggested values for the coefficient of vertical subgrade reaction, k_s, as follows:

Consistency of Clay	Stiff	Very Stiff	Hard
Values of q_u, ton/sq ft	1-2	2-4	>4
(kN/m^2)	(96-190)	(190-380)	>380
Range of k_s, ton/cu ft	50-100	100-200	>200
(kN/m^3)	(15,700-31,400)	(400-62,800)	>62,800
Proposed values of k_{s1}, ton/cu ft (kN/m^3)	75 (23,600)	150 (47,100)	300 (94,200)

For horizontal subgrade reaction with stiff clays Terzaghi gave the equation:

$$k_h = k_{s1}/1.5b$$

where:

k_h = coefficient of horizontal subgrade reaction,

k_{s1} = basic value of coefficient of vertical, subgrade reaction (k_s) for a square area 1 ft in width, and

b = width of pile or drilled shaft.

Note that adaptation of the coefficient of subgrade reaction to fit the soil modulus, E_s, leads to:

$$E_s = k_h b$$

Ultimate Resistance. Reese (1958) provided models for the ultimate resistance to lateral movement of a vertical shaft by assuming that the clay around the pile fails as a sliding block or as a group of sliding blocks. The failure is assumed to be undrained, with

a shear strength taken equal to the cohesion, c, and the angle of internal friction, ϕ, equal to zero.

At some depth, the soil is assumed to flow around the pile in a condition of plane strain, unaffected by the ground surface. The blocks shown in Fig. 2.2 can each be considered as samples of unit height tested to failure in plane strain shear tests. As the pile moves forward, σ_1 approaches zero on block E. To fail block E, σ_2 must be approximately 2c. To fail block D, σ_3 must therefore be approximately 4c. Continuation of this line of reasoning leads to a value of 10c for σ_6. The pile must therefore overcome 10c on the front face plus 1c on each side, leading to a total force exerted by the pile segment on the soil during failure of:

$$p_u = 12cb$$

Failure near the surface can be modelled conceptually as shown in Fig. 2.3 by a wedge of soil of height H. This wedge offers resistance to lateral movement by the cohesion along its vertical sides and by its own weight. If the components of these forces in the horizontal direction are summed and differentiated with respect to H, the ultimate resistance per unit length of pile will be as follows:

$$p_u = b\left(\bar{\gamma}H + \alpha c \cot\theta + 2c \cot\theta + \frac{2cH \sec\theta}{b}\right)$$

where:

$\bar{\gamma}$ = submerged unit weight if the soil is below the water table and total unit weight if the soil is above the water table,

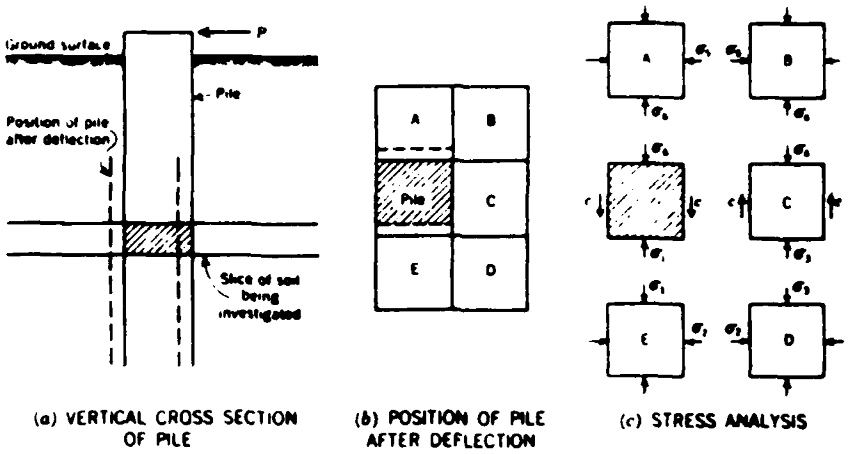


FIG 2.2 Analysis of Pile Deflection,

(after Reese, 1956)

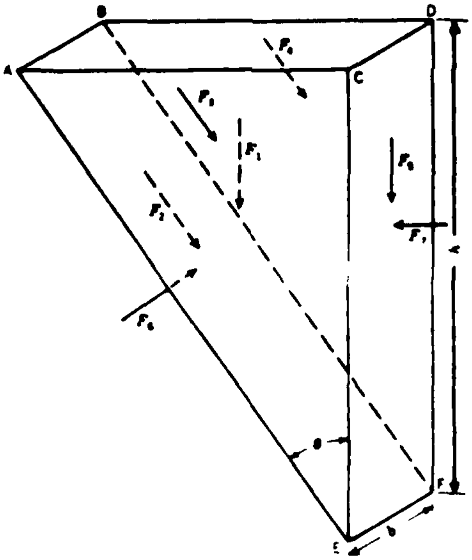


Fig. 2.3 Wedge of Soil Considered as a Free Body,

(after Reese, 1956)

α = reduction factor used in computing the vertical force exerted on the shaft along its length by the adhesion of the wedge, and

θ = angle defined in Fig. 2.3.

If the side friction along the pile is ignored and θ is assumed to be 45°, the equation above reduces to:

$$p_u = b\left(\bar{\gamma}H + 2c + Jc\,\frac{H}{b}\right)$$

where

$$J = 2 \sec 45° = 2.83$$

Note that according to these equations, the ultimate resistance at the surface is 2c. An ultimate resistance of 3c at the ground surface is typically assumed in existing methods for constructing p-y curves to correlate these equations with experimental results.

Obviously the models for ultimate resistance and initial soil modulus described above are crude simplifications of the way in which saturated clay would really behave in resistance to lateral loading. However, these models have been used as a point of departure for the derivation of design expressions from experimental results.

<u>Cyclic Loading</u>. Another advantage cited earlier leading to the use of the Winkler assumption with nonlinear p-y curves is the relatively straightforward way in which the effects of cyclic loading can be accommodated. Major experiments performed using repetitive loading have indicated that the ability of the soil to resist load can be dramatically reduced with increasing cycles of load. The relative importance of various factors contributing to this loss are not well

understood and remain the subject of much research. A good review of current thinking on the matter is provided by Long (1984). Factors known to be important include the gapping and scour which takes place immediately adjacent to the pile and the buildup of excess pore pressures from cyclically applied shear stresses. Each of the p-y criteria outlined by Reese (1984) provides a means of accommodating cyclic loading effects to force agreement with experimental results; however, the present lack of understanding of factors contributing to these effects makes it difficult to extrapolate these experimental results to other soil deposits or even conditions of loading other than those used in the tests.

Determination of Soil Properties for Design. Each of the specific p-y criteria described by Reese (1984) is based upon correlation of test results with some measure of undrained shear strength. It is a well established principle of soil mechanics that the shear strength of soil is a function of the effective stresses at failure (if in fact failure is definable). The use of an undrained analysis in a field loading situation thus requires that the tests used to predict the shear strength of soil accurately model the changes in effective stresses in the field. The following should be considered.

1. It is currently not possible to predict the state of stress around a pile.
2. The effect of disturbance (by pile installation) on the soil properties is not well known.

3. The loading conditions used in the typical undrained test methods do not match the orientation of principal stresses in the field.

The factors listed above certainly do not serve to enhance confidence in the use of the existing design criteria. In fact, the case could be made that the use of unconsolidated, undrained triaxial tests or vane shear tests to determine shear strength as recommended in several of the p-y criteria is little more than correlating p-y curves with a sophisticated index test. When using a recommended procedure for constructing p-y curves for a specific project, it is therefore of utmost importance to use the same type of tests and test conditions for strength measurement as was used in developing the criterion. The recipe also calls for a teaspoon of skepticism and a liberal dose of safety factor to give the cook a better night's sleep.

SUMMARY

The basic factors considered in the design of single piles for lateral loading have been summarized, along with a brief description of the method of analysis most commonly used for this situation. This chapter provides a basis of understanding of the laterally loaded pile problem from which to begin a discussion of the behavior of pile groups under lateral load.

CHAPTER 3

EFFECTS OF INSTALLATION ON THE BEHAVIOR OF GROUPS OF PILES UNDER LATERAL LOAD

INTRODUCTION

Research into the behavior of piles during axial loading has recently focused considerable attention toward the effects of installation on the piles and surrounding soil. While the effects of installation are recognized to influence pile response greatly during vertical loading, installation effects on the behavior of pile groups during lateral loading has frequently been neglected. Such neglect is only justifiable if an intelligent, rational assessment indicates that pile installation either has little influence or is inherently and correctly accounted for in the analysis. A rational assessment of installation effects requires a fundamental understanding of the behavior of the piles and surrounding soil during and after driving; although many aspects of this behavior are poorly understood in a quantitative sense, a review of concepts and observations should be useful at this point.

This chapter will provide a brief summary of available concepts and observations of the effects of pile installation on the behavior of piles and surrounding soils, with an emphasis on the influence of these effects on the performance of the pile group during lateral loading. The discussion will be restricted to driven piles in clay. This analysis will show that the effects of pile installation

on the behavior of groups under lateral loading are largely determined by the following factors:

1. displacement of the soil by a single pile,
2. driving sequence and spacing (group effects), and
3. in-situ soil properties (soil effects).

These factors will be shown to influence the behavior of pile groups by the following mechanisms:

1. by affecting the state of stress in the ground, and
2. by producing a change in properties of the clay.

The following sections of this chapter will discuss each of these mechanisms and consider the manner in which each is influenced by the aforementioned factors. Observations of the behavior of the Houston pile group will be included in the general discussion. A detailed discussion of the installation of this group has been presented by O'Neill, Hawkins, and Audibert (1982).

EFFECT OF INSTALLATION ON THE STATE OF STRESS IN THE GROUND

The forced entry of a pile into the soil will require displacement in some manner of a volume of soil equal to that of the pile. As soil is displaced away from the piles the direction and magnitude of the principal stresses will change, both temporarily and permanently. The design of piles for lateral loading is typically done utilizing some measure of undrained shear strength of the soil without consideration of effective stresses. It is important to understand something about the state of stress in the soil to ensure that the undrained-strength data used in the analysis are not grossly

misrepresentative of field conditions. The complexity of the problem in terms of both theoretical treatment and field observations has prevented a complete understanding of the problem. Nevertheless, several factors can be related to the stresses developed in the soil. Pile displacement, pile spacing, and in-situ soil properties are all important in regard to change in in-situ stresses.

Soil Displacement

Obviously, displacement-type piles such as closed-end pipe or precast concrete piles will have the most significant effect upon soil stresses, as indicated by well known problems with soil heave between and around the piles. Because the behavior of laterally loaded piles is most significantly affected by the soils within the top ten diameters, pipe piles driven open-ended may not produce large changes in in-situ stresses in the zone of most significance. Five or more diameters of penetration are often required for a plug to form in a pipe pile. Similarly, preboring may also significantly reduce stress changes in near-surface soils; however, Ray, Ulrich, and Malinak (1979) have observed substantial soil movements and possibly hydraulic fracturing of the soils during pile driving as a result of wet-rotary predrilling.

Relatively few attempts have been made to measure earth pressures against piles; O'Neill et al (1982) emphasized the difficulties in obtaining reliable measurements due to temperature sensitivity of the pressure cells, imperfect contact with the soil, and local soil anomalies. O'Neill measured earth pressures (total stress minus pore pressure) at depths of 20 to 40 ft which were approximately 3 times

the at-rest (K_0) pressures in a stiff clay. However, at a depth of 9 ft in a zone that was predrilled, stresses were approximately equal to at-rest pressures. Koizumi and Ito (1967) measured long-term stresses in a sensitive clay that were approximately twice the original vertical effective stress. Although these data are quite limited, they do tend to imply that the radial effective stress may likely be the major principal stress subsequent to pile driving.

Recent attempts to model pile installation as the expansion of a cylindrical cavity (Vesic, 1972; Randolph, Carter, and Wroth, 1979) have also tended to imply that the radial stress becomes the major principal stress after installation. However, the behavior of soils within the top 5 pile diameters may be influenced by the ground surface (boundary effects) to the extent that the cavity-expansion model is not a reasonable approximation of reality. Much additional work is needed before any conclusions beyond broad generalities can be made about the effects of pile displacement on the state of stress in the soil. The situation is further complicated by the effect of driving nearby piles.

Spacing of Piles in a Group

One would intuitively expect that large radial stresses around a pile would dissipate with increasing distance from the pile face, to a level eventually equal to the original at-rest pressure. However, piles driven in a closely spaced group would tend to produce stress overlap in the soil between the piles. Additionally, the existence of the relatively rigid, unyielding, previously driven piles may have an effect upon stress distribution around a pile as it is driven. There

exist no proven analytical models capable of evaluating these effects, but inferences can be drawn from the observations of Hagerty and Peck (1971). They observed lateral displacements after driving of as much as 1 in. for piles within a 6-pile group of 12-in -diameter piles spaced 3 ft on center in soft clay. Additional lateral displacements likely occurred during driving, but pile locations were not established until after completion of driving. All movements tended to be away from the most recently driven piles. Similar measurements by Bozozuk, Fellenius, and Sampson (1978) indicated lateral movements of up to 7 in. of some piles in large group of concrete piles driven in a soft, sensitive, marine clay. A conclusion could be made that the pile movements were in response to an increase in horizontal stress and proceeded as pore pressures dissipated from the outside in. The distribution of stresses would be expected to have an influence on the distribution of lateral load to the piles within a group, particularly in a group in which the piles were restrained against lateral movement soon after driving.

Observations by O'Neill (1983, personal communication) indicated less than 1/16 in. lateral movements of piles in a 9-pile group of 10.75-in.-diameter piles spaced 3 diameters on center in overconsolidated, fissured clay. Although predrilling to a depth of 10 pile diameters was used, measurements of heave indicate that considerable soil compression occurred during installation (which took 4 days). Therefore, while the installation of a group of closely spaced piles can be expected to affect soil stresses significantly, the properties of the clay appear to have an important influence as well.

In-Situ Soil Properties

Stress conditions in the soil after a pile is driven can be significantly affected by shear strength and sensitivity of the clay. Once again, the scarcity of data limits the strength of any arguments or conclusions, but implications can be drawn from observations.

The two well-documented cases of stress measurements on the piles mentioned earlier tend to indicate higher stresses in stiff, overconsolidated clays after driving. This trend is also predicted by the cavity-expansion model (Randolph, Carter, and Wroth, 1979; and Vesic, 1972). Intuitively, one would expect stiff clays to offer more resistance to displacement. Stiff clays typically attain their stiff character by overconsolidation; this stress history may result in coefficients of at-rest earth pressure greater than one. While original stresses in the ground have been thought to be important in relation to final stresses (Flaate, 1972), the model of Randolph et al indicates that the stress changes, normalized by the initial value of undrained shear strength are relatively independent of the ratio of maximum past pressure to vertical effective stress (OCR). The limitations of the Randolph model with regard to near-surface soils have been addressed. It can presently be stated only that the effects of overconsolidation and at-rest earth pressures on stress in soils near the ground surface after a pile is driven are uncertain, but appear less significant than the effects of undrained shear strength. Certainly several factors are interrelated.

The differences in behavior of soft and stiff clays during the driving of a pile is at least partly related to the dilatancy of

stiff, overconsolidated clays during shear. Sensitive clays exhibit an entirely different behavior during shear due to a breakdown of the clay structure. A thin zone of clay near the pile has been observed to become liquid and be extruded to the surface adjacent to the pile (Hagerty and Peck, 1971; and Legget, 1950). This extrusion can result in smaller lateral displacements and stresses.

EFFECT OF INSTALLATION ON SOIL PROPERTIES

The sometimes drastic changes in the state of stress in the ground discussed in the previous section can be expected to produce changes in the properties of the clay. Design of pile groups in clay for lateral loading is almost universally done on the basis of tests on clay in a relatively "undisturbed" state; that is, tests are performed on undisturbed samples or are performed in situ prior to pile driving. An understanding of the potential changes in soil properties during and after driving is important to understanding and predicting behavior of a pile group during subsequent loading. The following sections review the effect on soil properties after driving as influenced by displacement of an individual pile, pile-group spacing, and in-situ soil properties.

Soil Displacement by a Single Pile

Casagrande (1932) was one of the first to examine the effect of pile driving on the properties of clay. He concluded that the clay would be completely remolded within a distance of 0.5 diameters of a displacement pile and largely remolded within a distance of 1.5 diameters. This conclusion, together with his research into the proper-

ties of remolded clays, resulted in concern that disturbance caused by pile driving might actually result in increased settlement of structures supported by friction piles in soft clay relative to that expected for spread footings. Measurements by Cummings, Kerkhoff, and Peck (1950) of the shear strengths and water contents near piles after periods of up to eleven months after driving revealed an increase in shear strength and a decrease in water content in this zone of remolded soft clay. A lively debate ensued regarding the relative merits of friction piles in reducing settlements. Rutledge (1950) proposed that the increased strength could be explained by the following sequence of events (see Fig. 3.1).

1. The clay undergoes a reduction in strength upon remolding at a constant water content due to increased pore pressures and reduced effective stress.
2. The clay consolidates along a line which is roughly parallel to the virgin consolidation curve (plotted as water content versus logarithm of effective stress); the original strength is regained at a lower water content.
3. Increased stress from pile installation consolidates the clay to a higher effective stress and subsequently to a higher shear strength.

Subsequent measurements in soft clay by Seed and Reese (1955) and Flaate (1972) have shown similar effects. Measurements made by Seed and Reese are shown in Fig. 3.2 and typical results observed by Flaate are shown in Fig. 3.3. In-situ measurements of shear strength near a pile in sensitive clays after consolidation show that the shear

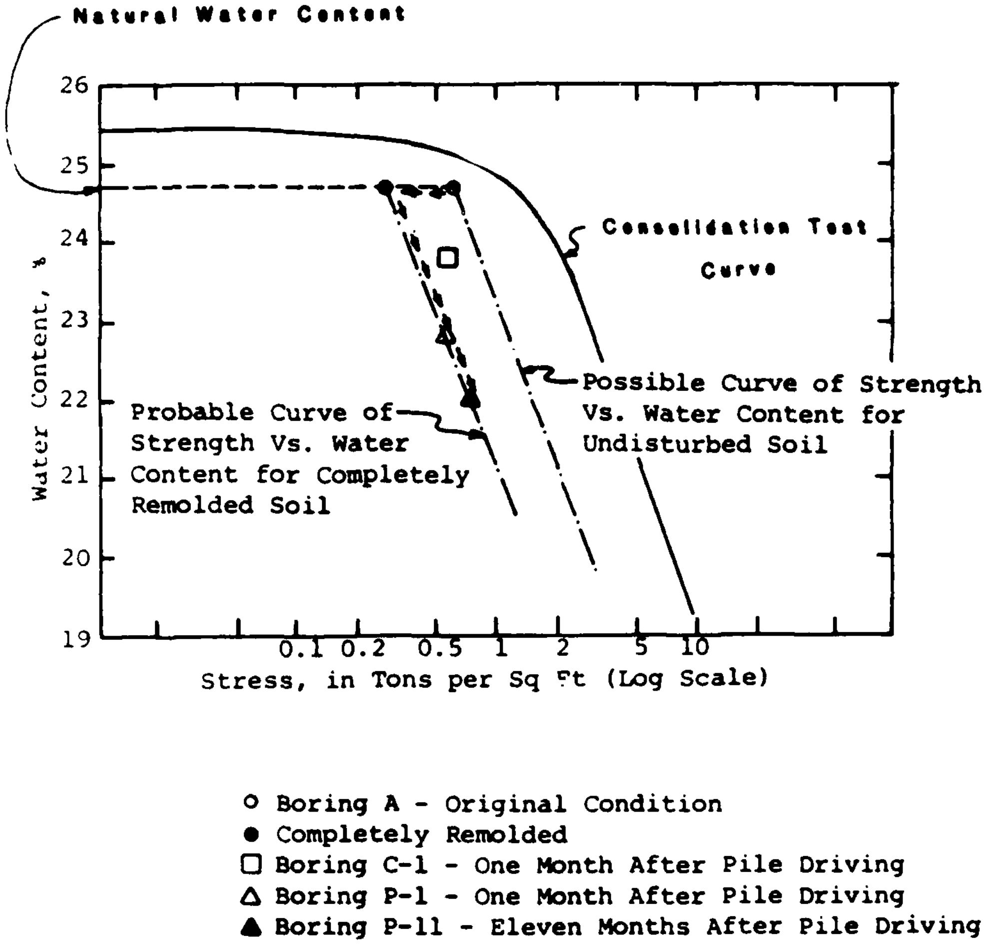


Fig. 3.1 Shear Strength vs Water Content after Pile Installation (after Rutledge, 1950)

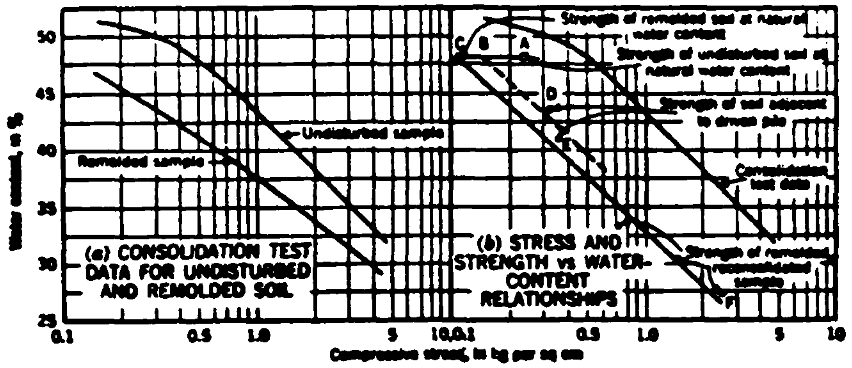


Fig. 3.2 The Relationship Between Water Content and Compressive Stress (after Seed and Reese, 1955)

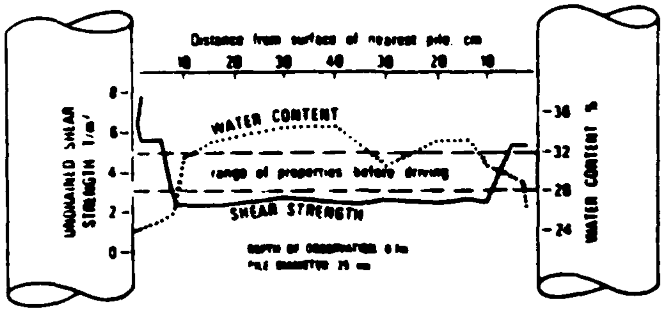


Fig. 3.3 Shear Strength and Water Content Variations Between Two Driven Piles (after Flaate, 1972)

strength ranges from a reduction of 15% from the original to a slight increase (Orrje and Broms, 1967; Bozozuk et al, 1978; and Roy et al, 1981). Decreases in shear strength outside the zone of remolding have been noted in many of the aforementioned references.

Although the most drastic changes in long-term strength and moisture content were noted within 1/2 diameter of the piles in the aforementioned studies, initial measurements of excess pore presure near a driven pile tend to indicate a consistent high pore pressure within a zone 2 to 4 diameters from a displacement pile, the larger distances in more sensitive clays (Roy et al, 1981; Lo and Stermac, 1965; and Hagerty and Garlanger, 1972). Excess pore pressures were noted in those studies to distances of 10 to 15 diameters away from a pile.

Soderberg (1962) has applied consolidation theory to the dissipation of pore pressure around a pile in an attempt to explain the time rate of the gain in bearing capacity of friction piles in clay. His analyses suggest that the time needed for consolidation around a pile should be proportional to the square of the pile diameter. Data collected by Vesic (1977) on the increase of bearing capacity with time for friction piles in clay are reproduced in Fig. 3.4. The points shown for the Horton Quay site represent piles driven in a group. If one believes that increase in capacity is directly related to dissipation of pore pressure, these data suggest that years may pass before excess porewater stress is lowered to an insignificant value. O'Neill (1983) has summarized measurements of dissipation of pore pressure around pile groups (see Fig. 3.5). It is clear from

Type	Dia.	Length ft.	Soil type	Location	Source
□ ■ steel H	14	(119) (219)	silt	Tappan Zee, N.Y.	Yang 1956
△ steel pipe	6	22	soft clay	San Francisco	Seed & Reese, 1957
▲ steel pipe	12	60	soft clay	Michigan	Housel 1958
● ● precast concrete	14	(40) (56)	soft boulder clay	Horten Quay	Bjerrum et al., 1958
● ● ○ steel pipe	24	(242) (316) (300)	soft to stiff clay	Eugene Island	McClelland, 1969
					Stevens, 1974 ---- (theoretical prediction)

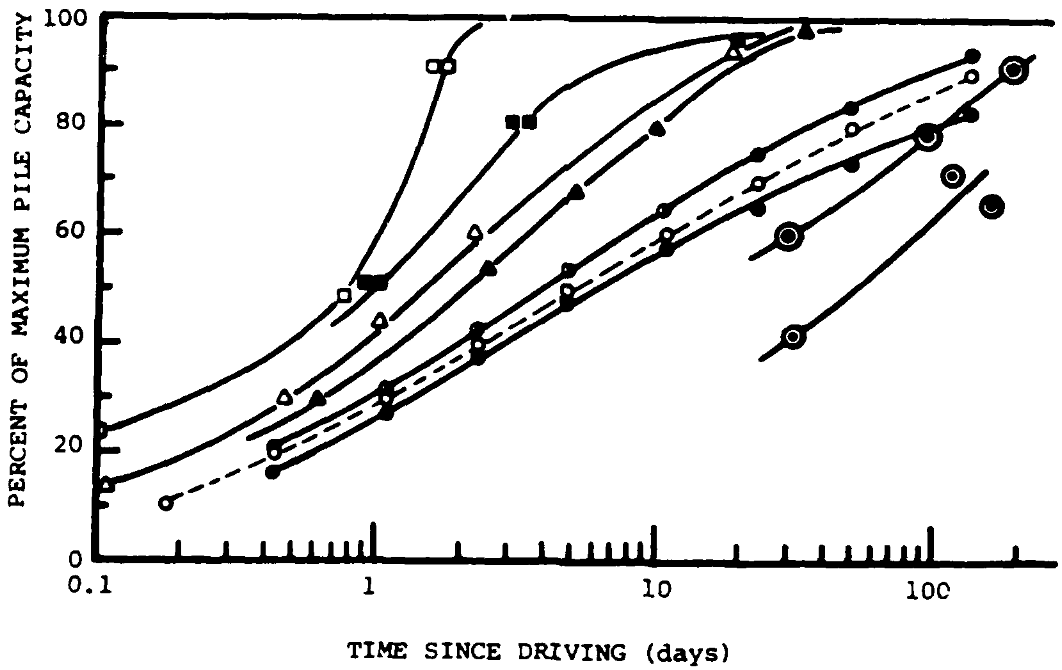


Fig. 3.4 Field Data on Increase of Bearing Capacity with Time for Friction Piles in Clay (after Vesic, 1977)

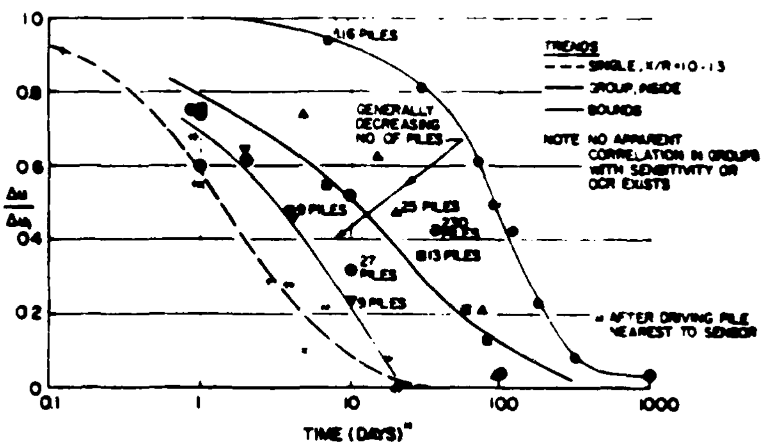


Fig. 3.5 Pore Water Pressure Dissipation Rates: Full-Scale Single Piles and Groups in Clay (after O'Neill, 1983)

these data that dissipation rates for pile groups are substantially slower than that of single piles. Within a pile group, much smaller initial-horizontal-pressure gradients exist than for a single pile.

Vesic (1972) presented equations for determining the extent of the plastic-shear zone using the theory of cavity expansion in an elastic-plastic soil. With an incompressible soil and $\phi = 0°$, these equations reduce to:

cylindrical cavity: $R_p/R_u = (E/3S_u)^{0.5}$

spherical cavity: $R_p/R_u = (E/3S_u)^{0.33}$

where: E = modulus of elasticity,

S_u = undrained shear strength,

R_p = radius of plastic shear zone, and

R_u = ultimate radius of the cavity.

Using typical values of E and c_u suggested by Vesic, the following shear zones are indicated:

Type Failure	R_p/R_u
Cylindrical Cavity	3 to 17
Spherical Cavity	2 to 7

These values indicate an expected zone of remolding much larger than that in which significant changes in shear strengths and water contents have been measured.

Of great importance in the design of piles for lateral loading is the effect of pile installation on the stress-strain curve for a clay which had been remolded and reconsolidated. While most research-

ers have not considered this aspect in any detail (most of the strength measurements around piles have been made with in-situ vane devices), Flaate observed a significant softening (i.e., greater deflection at a given applied stress) in the stress-strain curves obtained in the unconfined compresssion test. Of particular interest was the fact that Flaate's test results were for samples that were as much as 1.5 diameters away from the nearest pile. The peak values of undrained shear strength for Flaate's tests appeared to be relatively unaffected by pile installation.

Unfortunately, all of the data reviewed to this point have been concerned with soil near the wall of a pile and at a substantial depth. The use of the data to predict behavior at the shallow depths necessary to understand piles subjected to lateral loading may be misleading. The effects of remolding near the ground surface may also be different; certainly the pattern of deformation is different in the top two or three diameters and may result in a larger area of plastic deformation at the ground surface than at depth. The nearness of the ground surface (where there is a zero vertical stress) suggests that the long-term, radial stress cannot be as high near the ground surface as at depth, so that the reconsolidation pressures may not be high enough to produce the strength increases near the piles as is often noted. Bozozuk et al (1978) measured very slight strength reductions with the in-situ vane below the 6 ft depth in a soft clay but noted larger reductions in the upper 6 feet. More research will be needed to achieve an understanding of the effects of pile installation on soil behavior, particularly with regard to effects on deformational

properties that are important to lateral-load response. It must also be noted that a large proportion of pile groups subjected to large lateral loads consist of open-ended-pipe piles; the extent of remolding in the top few diameters of depth may be significantly less for open-ended-pipe piles and piles which are installed in predrilled holes than would be expected for displacement piles.

Spacing of Piles in a Group

If a zone of remolded soil around a pile is appreciably affected by pile driving, then installation of closely spaced piles in a group could affect a significant proportion of the deposit. Additionally, effects on soil properties outside of any zone of remoldng could be magnified by the additive action of several nearby piles. The mechanisms discussed in the previous section may not be altered, but simply made more significant by the close spacing of piles in a group.

In-Situ Soil Properties

Virtually all of the significant data on the effect of pile installation on soil properties has been for soft clay, often soft clays that are sensitive. These are soils which tend to densify during shear, producing excess pore pressure in addition to that caused by increases in normal stresses. Stiff, overconsolidated clays may tend to dilate during shear, producing excess pore pressures that are negative. These negative pore pressures act to offset the positive pore pressures caused by increased normal stresses. The hypothesis can be made that in a zone where shearing is significant and final all-around stresses are not substantially increased (as may be the

case near the top of the pile), the net effect may be one of reduced shear strength and stiffness. There are no data available to substantiate this hypothesis, but the basic differences in behavior during shear of clays with different dilatancy and mineral characteristics imply that the effects of pile installation could be drastically different for different soils. Among soft clays, differences in sensitivity resulted in different strengths after full consolidation, ranging from a slight reduction in shear strength to a substantial increase. Even though pile-installation effects cannot presently be quantified, the principles established by a consideration of these effects have implications regarding interpretation of results of pile-load tests. Interpretations of pile-load tests currently form the basis for design of pile foundations for both axial and lateral loads.

CONCLUSIONS

A consideration of the effects of pile installation on behavior of groups of driven piles in clay soils that are subjected to lateral loads leads to the following conclusions.

1. Installation of a pile group results in a change in the state of stress in the ground such that the radial effective stress is likely to be the final major principal stress. The change in stresses is more significant under the following conditions:

   a. when the piles are of a type to displace a large volume of soil,

b. when the piles are closely spaced; "closely spaced" may be on the order of 3 pile diameters or less, depending upon pile displacement, and

c. when the soil has a high shear strength and can therefore mobilize a greater resistance to displacement.

Another result of stress changes in the ground which may be significant is the fact that piles can be "preloaded" in different horizontal directions as subsequent piles in the group are driven. This pattern of preloading can affect the distribution of loads to the individual piles as the group is loaded.

Another implication of changes in the state of stress in the ground is that testing for undrained shear strength and soil stiffness should ultimately reflect those changes. The effects of installation on stresses in the soil are not currently understood to a sufficient degree to account properly for these changes; changes in soil properties during installation are interrelated with stress changes.

2. Installation of the pile in a group can result in significant changes in soil properties. Evaluation of these changes requires consideration of the following points.

   a. Installation of a pile displaces and remolds a portion of soil near the pile; the extent of this remolding and the effect on soil properties is related to the displacement of the pile and the final state of

stress. Changes in stiffness may occur outside this zone of remolding.

b. Closely spaced piles act to intensify the installation effects on the soil between the piles.

c. The effects of remolding and stress change can vary widely with the type of clay and particularly with its behavior during shear.

The most significant aspect of installation effects on soil behavior is that the data from load tests that form the basis for designs may be misinterpreted. Because most designs are based on semi-empirical predictions of soil behavior that is matched to load-test results, installation effects on soil properties are inherently encompassed in these predictions. A better understanding of installation effects could help prevent misapplications of design guidelines.

CHAPTER 4

MECHANICS OF PILE GROUPS UNDER LATERAL LOAD

INTRODUCTION

This chapter provides a discussion of the behavior of pile groups under lateral load. Some of the major design considerations for pile groups are outlined and pile-group behavior under lateral load is contrasted with that of single piles. The fundamental assumptions of several analytical models typically used in design are reviewed. The reader is assumed to be familiar with the mechanisms governing the behavior of single piles under lateral load and to be aware of commonly used design procedures.

Wright (1982) classifies pile-group problems into two general categories: those involving groups where the piles are widely spaced, and those involving groups where the piles are closely spaced. Groups of widely spaced piles are defined as those in which the individual piles interact only through the pile cap connection. Given that the response of the individual piles can be estimated for various conditions, the pile-group problem is a structural problem only. Groups of closely spaced piles are defined as those in which the response of an individual pile is influenced through the supporting soil by the response of other nearby piles, a situation commonly termed "pile-soil-pile interaction." This latter condition is the more dif-

ficult to quantify and is the subject of this research; groups of widely spaced piles will not be discussed further.

DESIGN CONCERNS

For a given loading condition, there are four major factors of importance to the design of pile groups.

1. The load causing collapse of the foundation must be known in order to establish an acceptable margin of safety.
2. The load-deflection response of the group as a whole is important for the design of the superstructure.
3. The distribution of load to the piles in the group is important both in predicting stresses and in the design of an efficient pile-group geometry.
4. The distribution and magnitude of stresses within the foundation piles is important for the design of the foundation.

COMPARISON WITH SINGLE PILE BEHAVIOR

There are several factors of importance to understanding the behavior of pile groups which are either less important or nonexistent for single piles. These factors include the effect of a pile cap or template (which may be embedded), the likelihood of greater top end restraint for piles in a group rather than for a single pile, the effect of the installation of nearby piles on the supporting soil, and the effect of pile-soil-pile interaction during loading.

Pile Cap

An embedded pile cap or template can undoubtedly have a significant effect on the performance of the pile group. The contribution of the cap or template to the ability of the group to carry lateral load is typically ignored for several reasons.

1. The soil surrounding the pile cap may not be in close contact with the cap throughout the life of the structure due to settlement after pile installation, scour, or other factors.
2. Ignoring the contribution of the pile cap is generally considered as conservative.
3. Commonly used design methods cannot model the behavior under lateral load of a pile group with an embedded cap.

Of these three factors, the first one is considered to be most important. The contribution of an embedded pile cap to the lateral load resistance will not be considered as a part of this study.

End Restraint

The pile cap or template will control the boundary conditions at the top of the piles, and will typically provide more rotational restraint than is present in single pile installations subject to lateral loading. While this rotational restraint can properly be accounted for in some analytical techniques, many only consider the extreme cases of "fixed" or "pinned" pile-head conditions. Neither of these extremes are ever precisely realized in practice, especially the fixed-head case. The boundary conditions at the top of a pile must be

considered in comparing results from an experiment with a single pile to those of a pile group, a fact which is sometimes overlooked (Matlock and Foo, 1976).

Installation

The installation of a group of piles in a clay soil is known to have an effect on both the properties of clay and the state of stress in the ground. While these changes certainly occur in single-pile installations, the effect is more significant in the case of pile groups. While installation effects have been a subject of much debate for many years, the geotechnical-engineering profession presently cannot offer much quantitative guidance to designers concerning the effect of pile installation on the behavior of pile groups under lateral load. A qualitative discussion of the current understanding of pile-installation effects is presented in Chapter 3 of this report.

Pile-Soil-Pile Interaction

In addition to the alteration of soil properties and stress states, the installation of closely spaced piles will affect the soil displacements and failure zones around piles that are loaded simultaneously. O'Neill (1977) has termed these as "mechanical effects" as opposed to "installation effects," although the two are acknowledged to be interdependent. The effect of interaction between piles during lateral loading is probably the most significant factor affecting pile-group behavior as compared with single piles, and is the primary focus of this research. The uncertainty in evaluating these effects is reflected in the significant differences in the approach to the problem used by the available analytical models. O'Neill states that

most models "are directly concerned with only that portion of the mechanical component of interaction that considers strain superposition in the soil mass and neglect both installation effects and failure-zone alteration." Consequently, "considerable judgement is required to design and analyze closely spaced groups." The experimental basis for this judgement is not at all well developed. The paragraphs which follow provide a review of several commonly used analytical models for groups of closely spaced piles subjected to lateral loads. These models should provide an overview of current thinking on the mechanics of pile-group behavior under lateral loads.

ANALYTICAL MODELS

O'Neill (1983) identifies five general classes of analytical models for groups of closely spaced piles:

1. finite element model,
2. continuum model,
3. modified continuum model,
4. modified unit load transfer, and
5. hybrid model.

The finite-element model is quite complex as a tool for practical problems and requires better soil data than is commonly available. An investigation of the results of the experiments described herein using finite-element techniques may prove to be rewarding; however, the finite-element method will not be included in the current study.

The continuum model is typified by the approach of Poulos (1971). Poulos assumes the soil to consist of an elastic medium and uses Mindlin's three-dimensional-elasticity equations to relate the soil reaction at a particular zone of one pile to all of the similar zones on other piles. The computer program DEFPIG has been used in several published papers (e.g., Poulos and Randolph, 1983) and was used to analyze the results of the study reported herein.

The modified continuum model (or coupled Winkler model) is similar to the continuum model except that the pile-soil-pile interactions are assumed to occur only in horizontal planes, thus reducing the computational effort significantly over the three-dimensional continuum. Examples of modified continuum models include those of Randolph (1980) and Nogami and Chen (1984). No modified continuum models were used as a part of the current study.

A modified-unit-load-transfer model is described by Bogard and Matlock (1983). This method is a procedure for constructing nonlinear load-transfer (p-y) curves for piles in a group by considering the piles within the group and the encompassed soil to act as one large-diameter pile. Soil resistance (p-y) curves for this large "imaginary pile" are used along with individual p-y curves to derive p-y curves for the group piles. Reese (1984) has also discussed the use of a single pile of a diameter encompassing the pile group as a possible bound on the behavior of the group. Both of these methods have been used in analyzing the experimental results of this study.

The hybrid model was initially proposed by Focht and Koch (1973), and has been widely used in the offshore industry. The

Focht-Koch approach utilizes the experimentally-derived, nonlinear, load-transfer curves to model the possible plastic deformations near the individual piles and the elasticity-derived interaction factors of Poulos (1971) to model the pile-soil-pile interaction. Group deflections are computed as the sum of the individual pile deflections plus the elastic deformation of the group. The p-y curves are then modified to force agreement with the group deflections. The resulting pile response using these p-y curves is taken to be in approximate agreement with the most heavily loaded piles. A similar but more refined approach was taken by O'Neill, Ghazzaly and Ha (1977) in developing the program PILGP1. Rather than using general pile-interaction factors, PILGP1 divides all of the piles into discrete elements and represents each as a Mindlin point load in computing interaction. The deflection at the location of each p-y curve on each pile is then computed from the load of all other pile elements and each p-y curve is individually modified for the effect of other piles. The response of each pile is recomputed using the modified p-y curves and the process repeated. The results of the experiment reported in Chapter 6 have been analyzed by both the Focht-Koch procedure and by PILGP2R, a later modification of PILGP1.

Within the five general types of analytical models for pile groups, specific procedures other than those mentioned here can be found. However, the procedures that are described above are the most widely used design methods and are representative of the different models. Repeated below are the specific methods of analysis that are employed in this study:

1. DEFPIG, the elasticity-based program developed by Poulos,
2. the single-pile method, assuming the group to act as a single large diameter pile,
3. the Bogard-Matlock procedure for generating modified load-transfer curves,
4. the Focht-hybrid method, and
5. PILGP2R, the hybrid program developed by O'Neill et al.

The paragraphs which follow present a more detailed discussion of each of the analytical methods that were employed.

DEFPIG

The program DEFPIG evolved from the early work of Poulos (1971), who presented solutions for the elastic analysis of the displacement of a pile subjected to lateral load and moment. The solution of a single pile under lateral load in an ideal, homogeneous, isotropic, semi-infinite, elastic mass involved the integration of Mindlin's equation (1936) for a horizontal load within such a semi-infinite mass. Soil displacements were evaluated by integrating Mindlin's equation over twenty-one rectangular elements used to represent the pile. Pile displacements were obtained from the equation of flexure of a thin strip, expressed in finite difference form. The solution was obtained by equating soil and pile displacements at the element centers. The solution was extended to pile groups in terms of two interaction factors, which are defined as the relative increase in deflection (or rotation) of a pile (at the groundline) due to another pile. The computation of these interaction factors is made in a manner similar to that as for a single pile, except that the soil dis-

placement must include the additional displacement due to the nearby pile. Interaction factors and individual pile stiffnesses for axial load are computed similarly. The analysis performed in DEFPIG proceeds as follows.

1. Interaction factors are computed for a range of pile spacings.

2. Interaction factors are computed for each pile in the group using the results of 1) and interpolating as necessary.

3. The deflection (in the axial and lateral directions) and rotation of a single pile are calculated.

4. The global stiffness matrix and load vectors are assembled for solution of:

   $$[A]\{P\} = \{\sigma\}$$

   where:

   $[A]$ = global stiffness matrix, order $3n \times 3n$,

   $\{P\}$ = vector of pile loads and moments, order $3n$,

   $\{\sigma\}$ = vector of pile head deflections and rotations, order $3n$, and

   n = number of piles.

   The stiffness of each pile in the axial, lateral, and rotational modes of displacement for axial and lateral loads and overturning moments are computed, using the results from step 3 and the interaction factors for each pile.

5. The equations of part 4) are modified for boundary conditions and solved.

Several modifications to the basic solution described previously are allowed in DEFPIG. Limiting values of pile-soil stress can be specified for each element to allow some nonlinearity in the solution for the single-pile computations. Displacements and rotations of a single pile, if known or previously computed, can be input as data. Nonhomogeneous soil profiles are approximately taken into account by making the displacement at a point dependent on the values of moduli at that point. The additional displacement due to interaction is computed using the average modulus between the two points of load. Note that the above modifications violate the assumptions of elasticity. The interaction factors are computed using the elastic solution regardless of such modifications.

Single-Pile Method

The single-pile method of analysis described by Reese (1984) is basically a means of establishing a bound on the effect of pile interaction for a group of closely spaced piles. By assuming that the soil contained within the piles moves with the group, the piles and enclosed soil are treated as a single pile of large diameter and analyzed using existing subgrade-reaction models. The piles are all assumed to have the same deflection at the top and to have the same deflected shape. This method uses existing procedures for constructing p-y curves for the large diameter pile. It is of interest to note that the commonly-accepted p-y criteria are largely empirical and are

derived from load tests on piles generally 24 inches in diameter or less.

The diameter of the large imaginary pile is taken as the circumference of the group divided by π. The stiffness is determined as the sum of the stiffness of the individual piles; note that the soil within the group is ignored as contributing to stiffness. The p-y curves for the imaginary pile are computed and an analysis is performed using a program such as COM622 (Reese, 1977) or BMCOL76 (Bogard and Matlock, 1977). The shear and moment for the imaginary large-sized pile is then distributed to the individual piles according to the ratio of the lateral stiffness of the individual pile to that of the group. The results of this solution are compared to that of a single pile analysis and the worst case (normally the group solution) is used for design.

Bogard-Matlock Method

The Bogard-Matlock (1983) method of analysis evolved from a reevaluation of the results of a series of static and cyclic lateral-load tests on circular pile groups at Harvey, Louisiana (Matlock, Ingram, Kelley, and Bogard, 1980), and was first presented formally in 1983. This method basically is a procedure for constructing nonlinear p-y curves for use with a Winkler-type soil model. Although the application of the method involves a substantial use of empirical factors (used to correlate with field data), the method is founded upon a logical hypothesis for group action as described below.

Bogard and Matlock conceptually approach the problem of describing the behavior of closely spaced pile groups by considering

zones of plastic deformation near each pile. When piles are spaced closely, these zones will tend to overlap and alter the plastic flow zone towards that of one large zone around the group (see Fig. 4.1.). In considering the extremes of pile interaction, piles that are widely spaced would result in the zones around individual piles being largely unaffected by other piles. The behavior of each pile could therefore be described using the current American Petroleum Institute (API) recommendations for p-y curves. For a group in which piles were close together, the group would tend to behave like a large pile with a stiffness equal to that of the sum of the individual pile stiffnesses and with a diameter equal to that of the group. This large pile could conceivably be analyzed in a similar manner, using p-y curves for a large diameter pile. The Bogard-Matlock procedure described below is a procedure for estimating the response for pile groups between these two extremes.

For a circular pile group spaced at intervals greater than one pile diameter on center, Bogard and Matlock propose that the lateral resistance at a given deflection for a large-diameter, imaginary pile encompassing the group may be distributed equally among all the piles in the group. The ultimate lateral resistance against each pile is then taken as the lesser of this distributed lateral resistance or that of each pile computed independently of the group. Bogard and Matlock suggest that the deflection of the piles in the group is related to both the deflection of the piles acting individually and the deflection of this large imaginary pile. They recommend computing

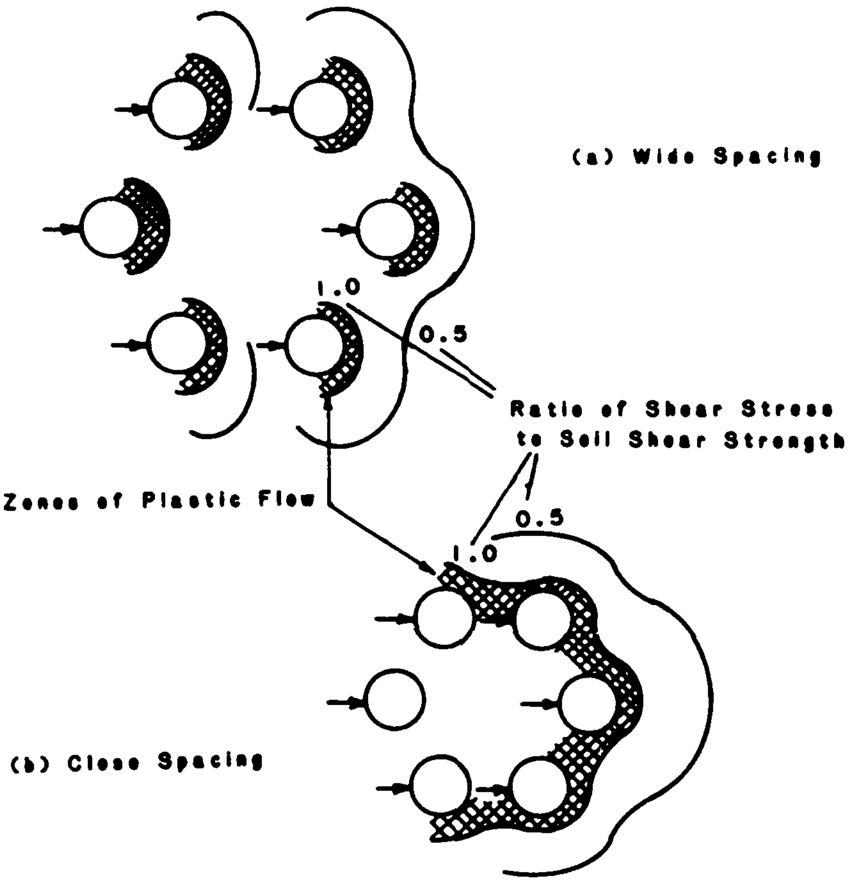


Fig. 4.1 Schematic Illustration of Pile Group under Lateral Loading (after Bogard and Matlock, 1983)

the deflection y for a given lateral resistance p by the following relationship.

$$y_{gp} = y_{sp} + (y_{ip}/s)$$

where:

y_{gp} = deflection of a group pile,

y_{sp} = computed deflection of an individual pile,

y_{ip} = computed deflection of the imaginary pile, and

s = pile spacing expressed in diameters.

These computed deflections are to be estimated using conventional, single-pile, p-y procedures at various lateral pressures to generate two p-y curves as shown in Fig. 4.2. With the maximum resistance limited to the lower peak of the two curves, these curves may then be used to analyze the pile group for bending on a per-pile basis using conventional single-member solutions.

One advantage of using the Winkler soil model with nonlinear p-y curves is that the effects of cyclic lateral loading have been accommodated in a relatively straightforward way. Bogard and Matlock attempt to incorporate the effects of cyclic loading in a manner similar to that used for static loads, but with some modifications. The results of the cyclic, lateral-load tests at Harvey apparently implied that the initiation of cyclic degradation in resistance occurred at a value of pile deflection related to the individual pile diameters, rather than to the diameter of the pile group. Additionally, the near-surface, cyclic degradation related to gapping and soil scour is not greatly affected by pile interaction. Bogard and Matlock there-

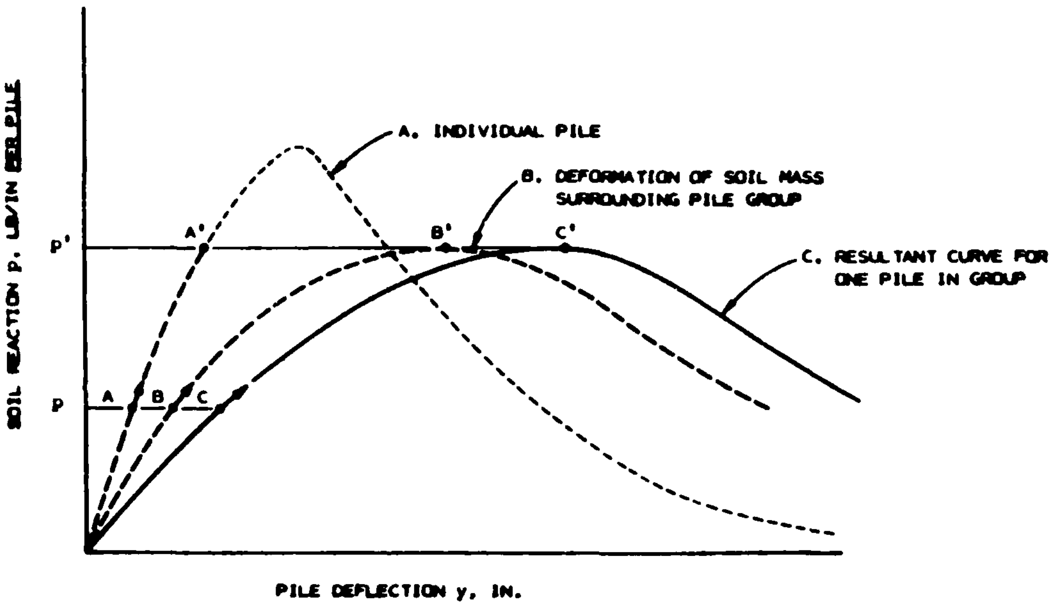


Fig. 4.2 Conceptual Construction of p-y Curves for Piles Within Groups (after Bogard and Matlock, 1983)

fore recommend approximating the degree of pile-group interaction by reducing the component of deflection from the imaginary pile near the surface. This reduction is accomplished using the pseudo-depth defined as x_r in the current single-pile criteria (Matlock, 1970), where x_r is related to the depth at which the presence of the free surface no longer influences the ultimate soil resistance against the pile. They propose that the component of deflection from the imaginary pile p-y curve be reduced by the ratio of the depth x to the term x_r computed at the same depth for the individual pile. This modification results in a gradual transition from no pile-group interaction at the surface to full interaction at and below the depth x_r. Although not stated specifically in their 1983 paper, it is apparent from their examples that Bogard and Matlock intend that static p-y curves for the imaginary pile should be used with cyclic p-y curves for a single pile to compute p-y curves for a pile group under cyclic loading.

Focht-Koch Method

Focht and Koch (1973) postulated that the pile-group behavior was controlled by two basically different stress conditions; a zone of high stress close to the pile which would result in large displacements and plastic strain in the soil, and a region of relatively small stresses and strains in the soil between the piles. They felt that these two different stress conditions could be analyzed separately by methods that have been found to be best suited for each. The p-y method using the Winkler model with nonlinear curves of soil response is used for describing the performance of individual piles, as this model inherently considers the high stress zone close to the pile.

Additional displacements for individual piles due to nearby piles are computed using elastic methods and the response of the group is predicted by assuming superposition of these computed displacements. In other words:

$$Y_G = Y_s + Y_g$$

where:

Y_G = pile group displacement,

Y_s = individual pile displacement, and

Y_g = displacement of a pile due to others.

For individual pile response, p-y curves are computed and used with a computer program such as COM622 or BMCOL76 to yield Y_s. This procedure is typically used for single-pile design and thus yields a basis for comparison of pile-group results.

The additional displacements from other piles are computed using elastic methods described by Poulos (1971). Poulos developed the following equation to obtain the deflection and load on each of the piles in a group, assuming the soil to act elastically.

$$\rho_k = \rho_F \sum_{\substack{j=1 \\ j \neq k}}^{m} \left(H_j \alpha_{\rho Hjk} + H_k \right)$$

where:

ρ_k = deflection of the k-th pile,

ρ_F = the unit reference displacement of a single pile under a unit horizontal load, computed by using elastic theory,

H_j = lateral load on pile j,

$\alpha_{\rho Hkj}$ = the coefficient to get the influence of pile j on pile k in computing the deflection ρ (the subscript H pertains to the case in which shear is applied at the pile head; there are also influence coefficients for applied moment and for the fixed head case),

H_k = lateral load on pile k, and

m = number of piles in group.

If the total load on the group is H_G, then

$$H_G = \sum_{j=1}^{m} H_j$$

For the usual assumption of a pile cap that is rigid such that each of the piles deflect an equal amount, the deflection ρ_k is equal to Y_G, the group deflection. The above equation can then be solved for the deflection and load on each pile. The influence coefficients α were presented in graphical form by Poulos; they can also be obtained using the program DEFPIG, previously described.

Focht and Koch's modification of the above procedure begins by introducing a term R, such that

$$\rho_k = \rho_F \sum_{\substack{j=1 \\ j \neq k}}^{m} \left(H_j \alpha_{\rho Hkj} + RH_k \right)$$

where:

R = relative stiffness factor, equal to the ratio of the mudline deflection computed by the p-y approach, Y_s, to the deflection ρ computed by the Poulos elastic method.

The above equation is used to solve for group deflection, Y_G, and pile head loads, H_j, as described previously. If an elastic modulus is employed that is representative of the relatively small strains and stresses expected for the region between the piles, the value of ρ

computed using the equations of Poulos normally will yield individual pile deflections smaller than that predicted by the p-y approach; thus, R is normally greater than one. The Focht-Koch modification will thus have the effect of distributing pile-head shears more evenly throughout the group than the purely elastic solution.

Perhaps the most desirable feature of the Focht-Koch procedure for designers is that the modification of p-y curves as described above for piles in the group will allow the computation of bending stresses as a function of depth. Focht and Koch recommend that the p-y data used for the response of an individual pile in the group be modified by the use of "Y" factors. The "Y" factors are constant multipliers used to increase the deflection values of each point on each p-y curve, thus generating a new set of p-y curves for each "Y" factor. These sets of p-y curves are used with the maximum shear load on a pile in the group to yield new values of mudline displacement for an isolated pile. The displacement values are plotted as a function of the "Y" factor, as shown in Fig. 4.3. A "Y" factor is selected that produces a displacement equal to that calculated for the group. The pile response (moments, stresses, displacements as a function of depth) that is computed by using the modified p-y curves, corresponding to the selected "Y" factor, is assumed to be representative of the most heavily loaded pile in the group. The design of the piles in the group is based on the behavior of the individual pile with the heaviest load.

Focht and Koch maintained that their procedure could account reasonably well for the effects of axial load and pile-head fixity,

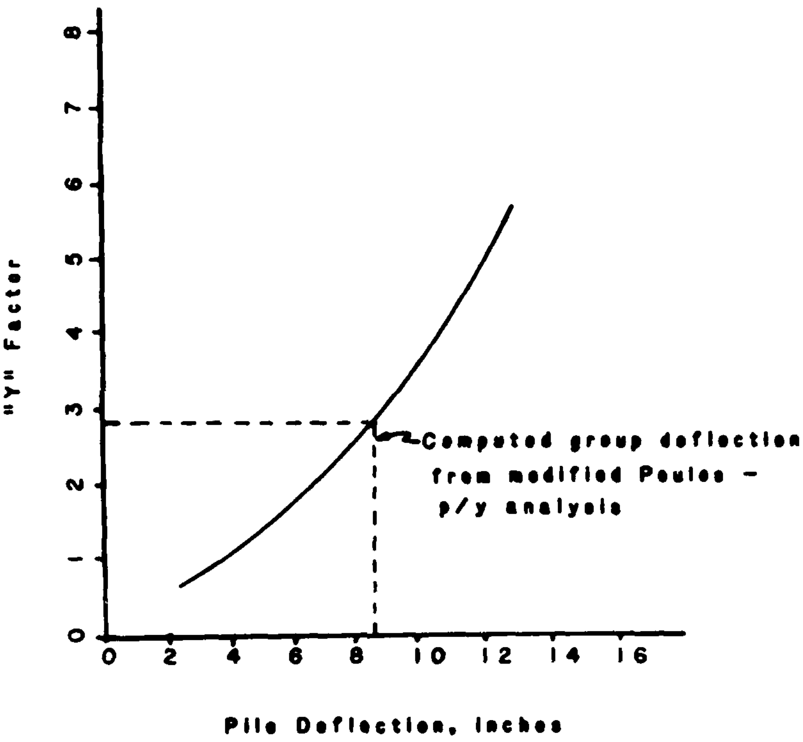


Fig. 4.3 "Y" Factor Influence on Computed Deflection (after Focht and Koch, 1973)

and could be used with p-y curves for cyclic loading to account for gapping and scour. They also suggested that "P" factors in the range of 0.7 to 1.0 might be used to account for shadowing effects of closely spaced piles.

PILGP2R

This program uses a hybrid-type model developed by Ha and O'Neill (1981) that is perhaps a logical extension of the earlier thinking of Focht and Koch (1973). PILGP2R was developed specifically to model three-dimensional pile geometry and loading at working loads and full load-deformation behavior in axially loaded groups of vertical piles. In addition to providing a complete solution for the behavior of all piles in a group in virtually any configuration, this model differs from the Focht-Koch procedure in that the unit-load-transfer curves are modified individually for the stresses from nearby piles. The earliest description of this method was published in 1977 by O'Neill, Ghazzaly and Ha; minor modifications to the program have been made and a 1981 publication by Ha and O'Neill provide a more up-to-date description. The code PILGP1 described in the 1981 article has been altered slightly to accommodate piles in a variety of soil conditions and the name has been modified to PILGP2R. The program PILGP2R was used as described later. Although PILGP2R is designed to accommodate general loading of pile groups having a variety of configurations (such as batter piles and piles socketed into rock), the following discussion will concentrate on the features of the code of importance to groups of vertical piles subjected to lateral load.

PILGP2R first proceeds toward a solution by neglecting the effects of pile-soil-pile interaction and follows a procedure similar to that described by Reese, O'Neill, and Smith (1970). A pile cap that is rigid is assumed and any effects of the soil acting directly on the cap are neglected. Each pile is then replaced (on the cap) by six nonlinear springs acting to resist displacement and rotation in three dimensions. For vertical piles these nonlinear springs are generated by an independent analysis for each mode of behavior as follows.

1. Horizontal displacement vs. horizontal shear and horizontal displacement vs. overturning moment in each of two directions are analyzed for a series of shear loads using the p-y approach described earlier; p-y curves are input by the user. Plots of these relationships are called "Mode I" curves by Ha and O'Neill.
2. Rotation in a vertical plane vs. horizontal shear and rotation in a vertical plane vs. overturning moment in each of two directions are analyzed using the p-y approach to produce "Mode II" curves.
3. Vertical displacement vs. axial load is analyzed using the load transfer or "t-z" approach similar to that described by Coyle and Reese (1966) to produce "Mode III" curves. The t-z curves are input by the user.
4. Torsional rotation vs. torsional moment or "Mode IV" curves are assumed to be linear. The slope (modulus) is taken as equal to JG/L where J is the polar moment of

inertia of the pile at the pile head, G is the shear modulus of the pile material, and L is one-half the pile length. Because torsional reactions are generally minor relative to the other components of pile reaction, the modulus is computed internally rather than input by the user.

The mode curves I, II, and III are fitted using cubic splines to obtain a smooth relationship. An inherent assumption made in using these mode curves is that there is no coupling between any of the aforementioned response characteristics; i.e., axial load does not affect behavior under lateral load, etc. Initial tangents to the mode curves are computed to allow initial representation using linear "spring constants" to assemble into the following equation.

$$[S]\{\sigma\} = \{F\}$$

where:

$[S]$ = stiffness matrix,

$\{\sigma\}$ = displacement vector, and

$\{F\}$ = force vector.

Using a geometric-transformation matrix, the stiffness matrices of the individual piles are summed and assembled into a global stiffness matrix for solution of the group displacements, and subsequently for determination of the individual pile-head displacements. Using the above equation, the pile-head reactions can be computed and checked for compatibility with the mode-curve-displacement vs. load relationship. In order to permit compliance with the nonlinear mode

curves, the load vector is applied in small increments and the pile head reactions and deformations are summed after each increment. Subsequent to the application of the first increment, the equivalent-spring constants are taken as a tangent modulus to each of the mode curves at the previously computed load. The p-y and t-z methods are used after solving for pile-head loadings to obtain stresses, displacements, and soil reactions with depth for each pile.

With the soil reactions now known as a function of depth for each pile, the procedure for computing pile-soil-pile interaction may be performed. The soil reaction over each pile increment is summed and replaced by an equivalent point load for purposes of using Mindlin's equations (1936) for point loads beneath the surface of a semi-infinite, elastic, half space. At the location of each p-y curve, the displacement of the soil mass at that location due to the point loads from all other piles is computed. Effects of a varying elastic modulus are handled by using the average of the values at the point of load and at the location in question. The p-y curve at that location is then "stretched" by multiplying all displacement values for that p-y curve by a factor B, where:

$$B = (\delta_i + \delta_m)\delta_i$$

where:

δ_i = displacement computed from noninteraction solution, and

δ_m = displacement from the sum of all other pile point loads.

If δ_m and δ_i are in opposite directions, then B is taken as:

$$B = \delta_i/(\delta_i + \delta_m)$$

The effect of this "stretching" is illustrated graphically in Fig. 4.4. All p-y curves on all piles (in each direction) are so modified, as well as t-z curves. In order to save expense in computing time, a radial distance, r, is typically specified beyond which the effects of any point loads are neglected. The final solution is then computed as described earlier for the noninteractive solution, but with the modified load-transfer curves. Several such iterations could be done, but at substantial additional expense in computer time; only one set of corrections is thought sufficient for design purposes. The overall flowchart for program execution is illustrated in Fig. 4.5.

Some readily apparent differences in results from PILGP2R and the Focht-Koch procedure follow logically. Because a constant y-multiplier is used in the Focht-Koch procedure to force a mudline displacement match with that of the group analysis, and because displacements are greatest at the mudline and progresively diminish with depth, the greatest offset of p-y curves (in an absolute sense) will occur at the mudline and diminish with depth. Due to the effect of the soil surface and the fact that the maximum soil resistance (and thus point load value) typically occur at some depth below the surface, the greatest offset of the p-y curves in the PILGP2R analysis will occur below the surface. Near-surface p-y curves in the PILGP2R method are actually little affected by group effects.

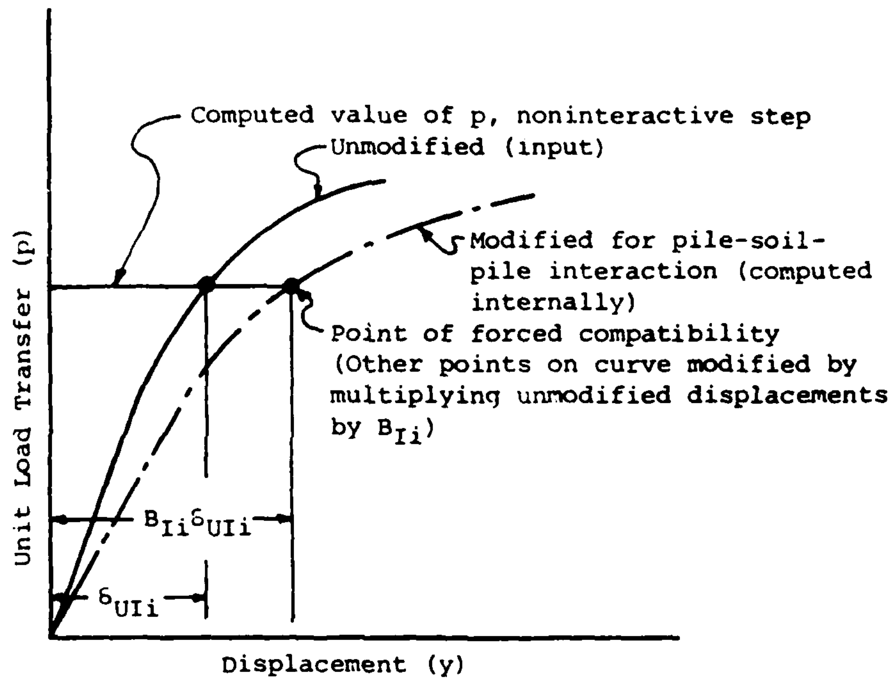


Fig. 4.4 Modification of Unit Load Transfer Relationship for Group Effects at Node i, Pile I (after Ha and O'Neill, 1981)

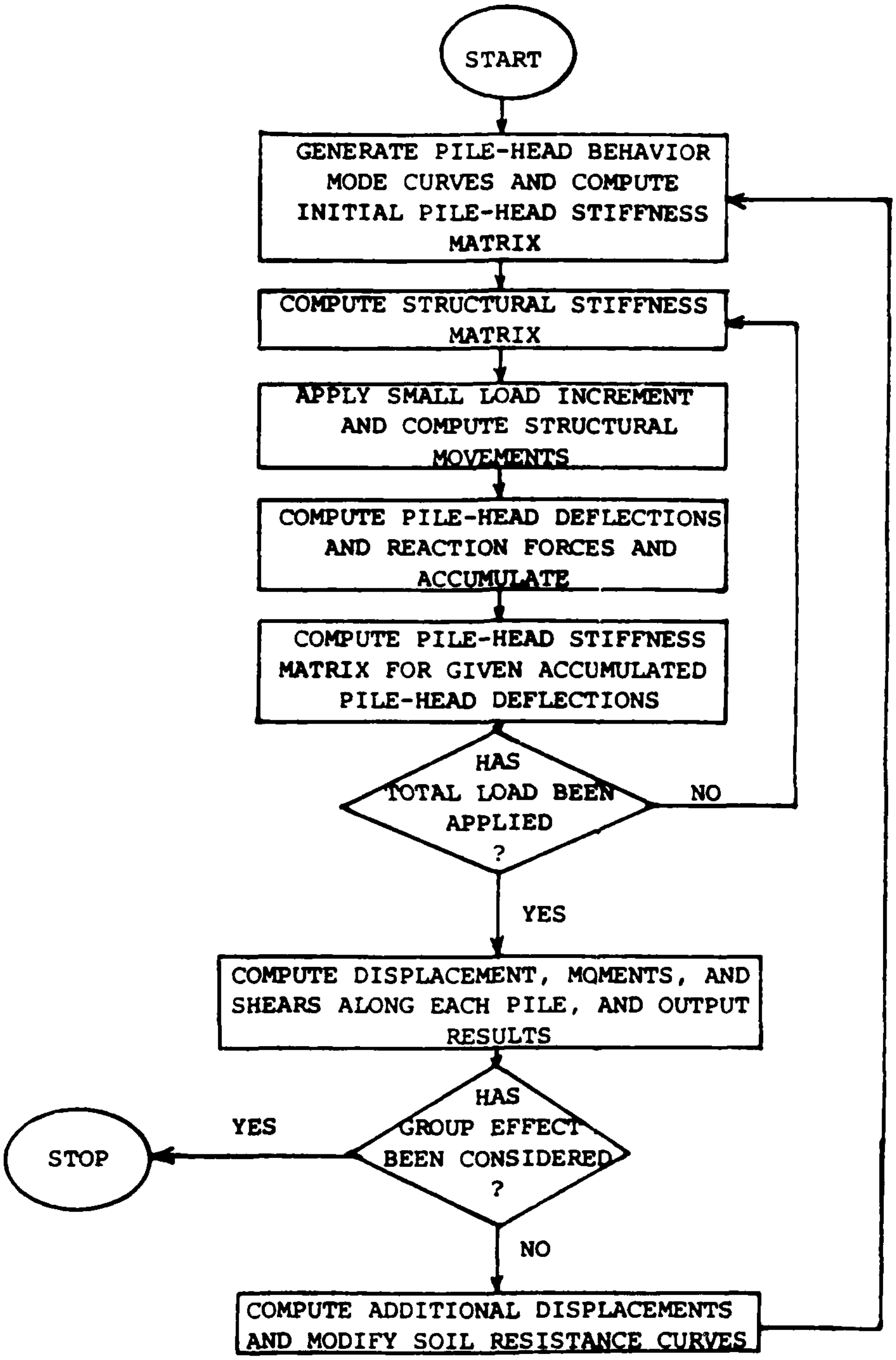


Fig. 4.5 General Flow Chart for PILGP1 (after Ha and O'Neill, 1981)

There currently exist no means of rationally accounting for a reduction in the ultimate p values used in p-y curves for pile groups with PILGP2R, and thus the authors do not recommend its use as a model to analyze groups with lateral loads beyond the working load range.

SUMMARY

This chapter contains a review of major design considerations for pile groups under lateral load and an examination of some of the analytical models most commonly used in design. This chapter has been intended to provide a basis for discussion of the experimental results presented in Chapter 7.

CHAPTER 5

PREVIOUS EXPERIMENTS WITH PILE GROUPS UNDER LATERAL LOAD

INTRODUCTION

According to O'Neill (1983), experimental results provide the principal sources of information that engineers must rely upon to develop judgement. This chapter offers a review of experimental work done prior to this study and, in so doing, places this experiment within the framework of the history of the subject. As will be seen, there exist very few full-scale experiments which were well instrumented and carefully performed, and none exist that allow actual determination of the distribution of soil resistance and pile stresses throughout the group to the extent attempted in the current study. Designs of pile groups for large lateral loads are presently based upon an extremely limited data base.

MODEL TESTS

Model tests provide an inexpensive means of conducting parametric studies for evaluating the effects of such factors as group geometry, pile spacing, and pile penetration. They are not so useful in developing design criteria. The most difficult problem with model studies is the inability to account correctly for the influence of the soil stresses that exist in the prototype. Effects of pile installation on stresses and soil disturbance is likewise a problem. While centrifuge modelling is thought by many to hold the promise of over-

coming some of these difficulties, the application of this tool to the problem of pile groups under lateral load has yet to be reported in the technical literature. Model studies will not be emphasized in this chapter.

FULL-SCALE TESTING

Full-scale tests currently provide the only measure of group behavior that can lead to the development of design guides that can be used with reasonable confidence. Yet, numerous problems must be addressed before even full-scale test results can be confidently accepted as representative of true field conditions. "Full-scale" tests may not actually be "full-scale" for the problem to which the results are being applied; many piles used in an offshore environment are several times larger in diameter than those used in any so-called "full-scale" experiment, and the effects of pile diameter on the behavior of even single piles are not well known. Full-scale tests are generally of lower quality than model tests that are conducted in a controlled environment, with instrumentation often influenced by harsh weather and careless construction crews. Soil properties may be variable and difficult to determine with a high degree of accuracy. The great expense and possibility of variations in soil conditions make parametric studies impossible, thus leaving the results subject to unlimited speculation about parameters which could have affected the results significantly.

In spite of the aforementioned problems with full-scale field tests, they provide the basis for design of piles subjected to lateral

loading. The paragraphs which follow will briefly describe a number of such tests, along with one model study in soft clay.

PILE-GROUP EXPERIMENTS

Feagin - Lock No. 25

Some of the earliest tests of pile groups under lateral loads were performed by the Corps of Engineers in the 1930s. The Corps had a massive construction project underway to improve navigability of the upper Mississippi River, and performed a large number of tests in conjunction with that construction. Tests of various arrangements of groups of battered and vertical piles with heads fixed in concrete monoliths were performed in the summer of 1936 at Lock No. 25 near Winfield, Missouri, and reported by Feagin (1955).

Soil information reported by Feagin indicates the soil to consist of fine to coarse sand containing occasional gravel. The general layout of the test monoliths is reproduced in Fig. 5.1. The piles were all of timber, driven to 30 ft of penetration and embedded 2 ft into the concrete cap. The pile caps were apparently cast upon the ground, but were not embedded.

Tests were performed under approximately static conditions by jacking apart the test monoliths. Various vertical loads were also used during testing. Gross loads and deflections of the monoliths were measured, and are summarized in Fig. 5.2. The monoliths were not tested to failure, and a maximum deflection of less than 0.5 in. was measured in most of the tests.

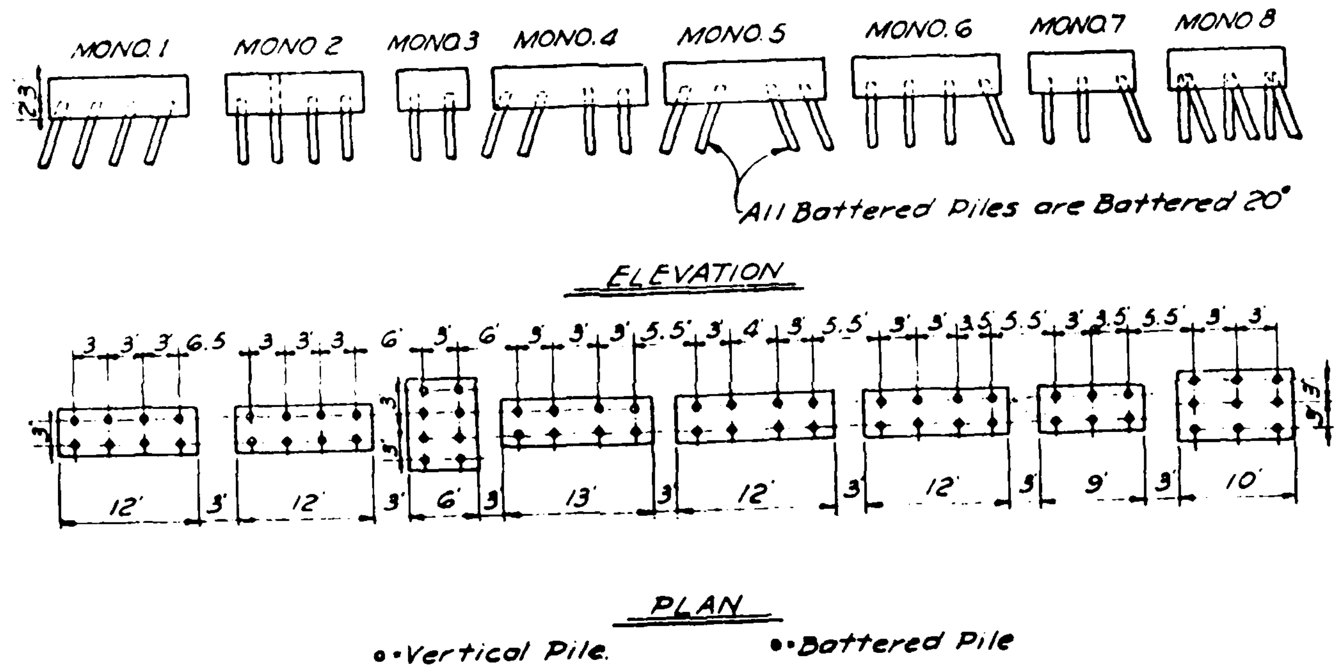
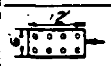
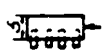
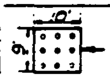


Fig. 5.1 Plan and Elevation of Test Monoliths - Lock 25 (after Feagin, 1955)

Monolith Number	Test Number	Schematic Diagram[a] Plan	Schematic Diagram[a] Elevation	Batter Ratio, per cent	Lateral Load for ¼-in. Deflection, tons per pile	Lateral Movements under Lateral Loads, in. 5 tons per pile	Lateral Movements under Lateral Loads, in. 10 tons per pile	Lateral Movements under Lateral Loads, in. Maximum per pile	Maximum Lateral Load Applied, tons per pile	Recovery after Maximum Load Released, per cent
3	1			0	4.8	0.270	...	0.492	7.5	61.3
2	2			0	5.8	0.197	0.660	0.790	12.0	54.4
6	3			25	7.0	0.143	0.455	0.455	10.0	71.8
8	4			33	7.1	0.116	0.482	0.482	10.0	50.1
7	5			33	7.3	0.130	0.420	0.443	10.5	71.1
4	6			50	9.0	0.076	0.270	0.345	11.5	86.1
1	7			100	9.0	0.071	0.310	0.489	12.0	91.6
4	8			50	11.0[b]	0.098	0.222	0.222	10.0	47.8
5	9			100	15.8	0.038	0.110	0.268	16.5	61.2

[a] ○ = Vertical Pile ● = Battered Pile   [b] Extrapolated

Fig. 5.2 Lateral Load Test Data - Lock 25 (after Feagin, 1955)

Manoliu et al - Danube River

Manoliu, Botea and Constantinescu (1977) reported a series of tests on 2- and 3-pile groups at a site along the Danube River in Romania. These piles consisted of 40-cm-square, precast concrete piles, driven to an embedment of 16.0 meters. The piles were embedded into a concrete cap which was not in contact with the foundation soil. Soil at the site in the region of most interest consisted predominately of medium silty clays with a cohesion from unconfined compression tests of about 700 lb/sq ft and moisture contents of 40 to 45%.

A sketch of the test layout is presented in Fig. 5.3. Loads were applied statically by hydraulic jacks and displacement of the pile caps was measured with dial gauges. A summary of the test results is provided in Fig. 5.4.

Beatty - Detroit Steel Mill

Beatty (1970) reported the results of a series of tests conducted on existing pile foundations of a structure that was being demolished and replaced. The program included tests on 6-pile groups and 2-pile groups of concrete-filled Raymond Step-Taper piles as shown in Figs. 5.5 and 5.6. Soil conditions consisted of soft clays and silty clays of high water content. Although no data were reported from laboratory testing, SPT (Standard Penetration Test) blowcounts were typically 2 to 3. The piles were embedded some 83 ft and their tips reached a layer of hardpan. Actual spacings between piles was not reported. All of the piles were embedded into massive concrete pile caps, some of which were excavated to eliminate passive soil resistance and some of which were not. Loads were essentially stat-

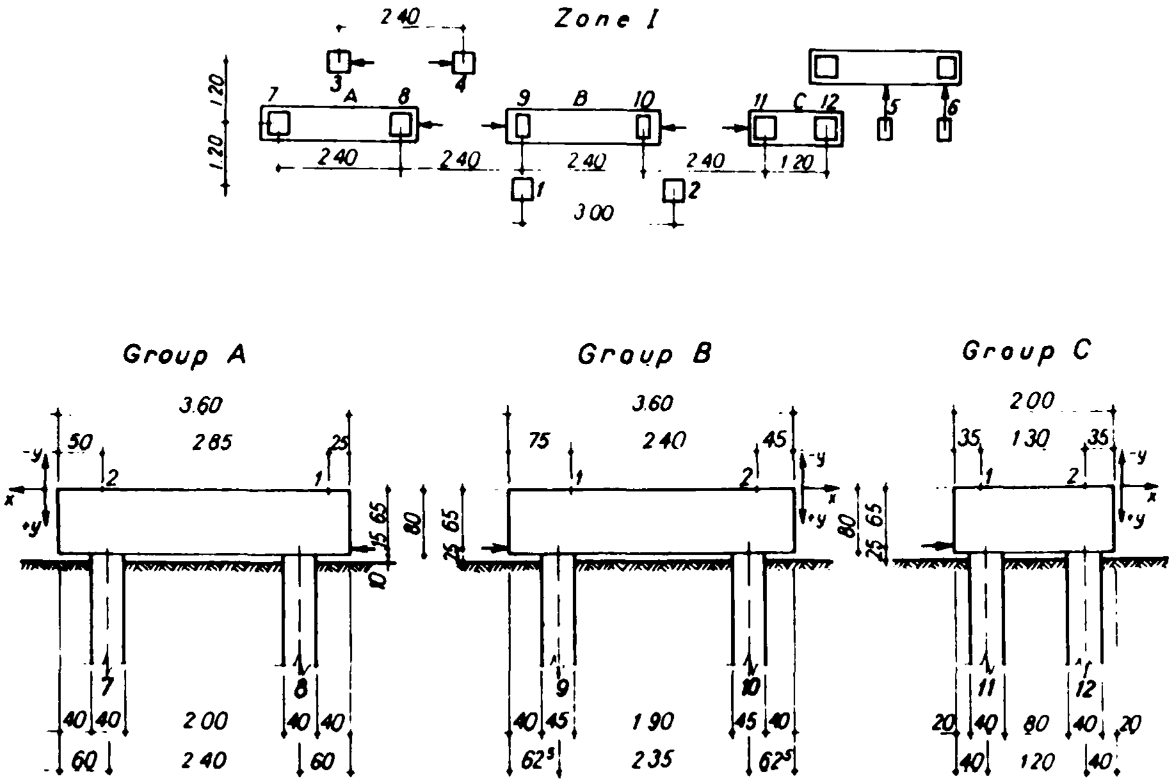


Fig. 5.3 Test Layout for Groups A, B, & C - Danube River (after Manoliu et al, 1977)

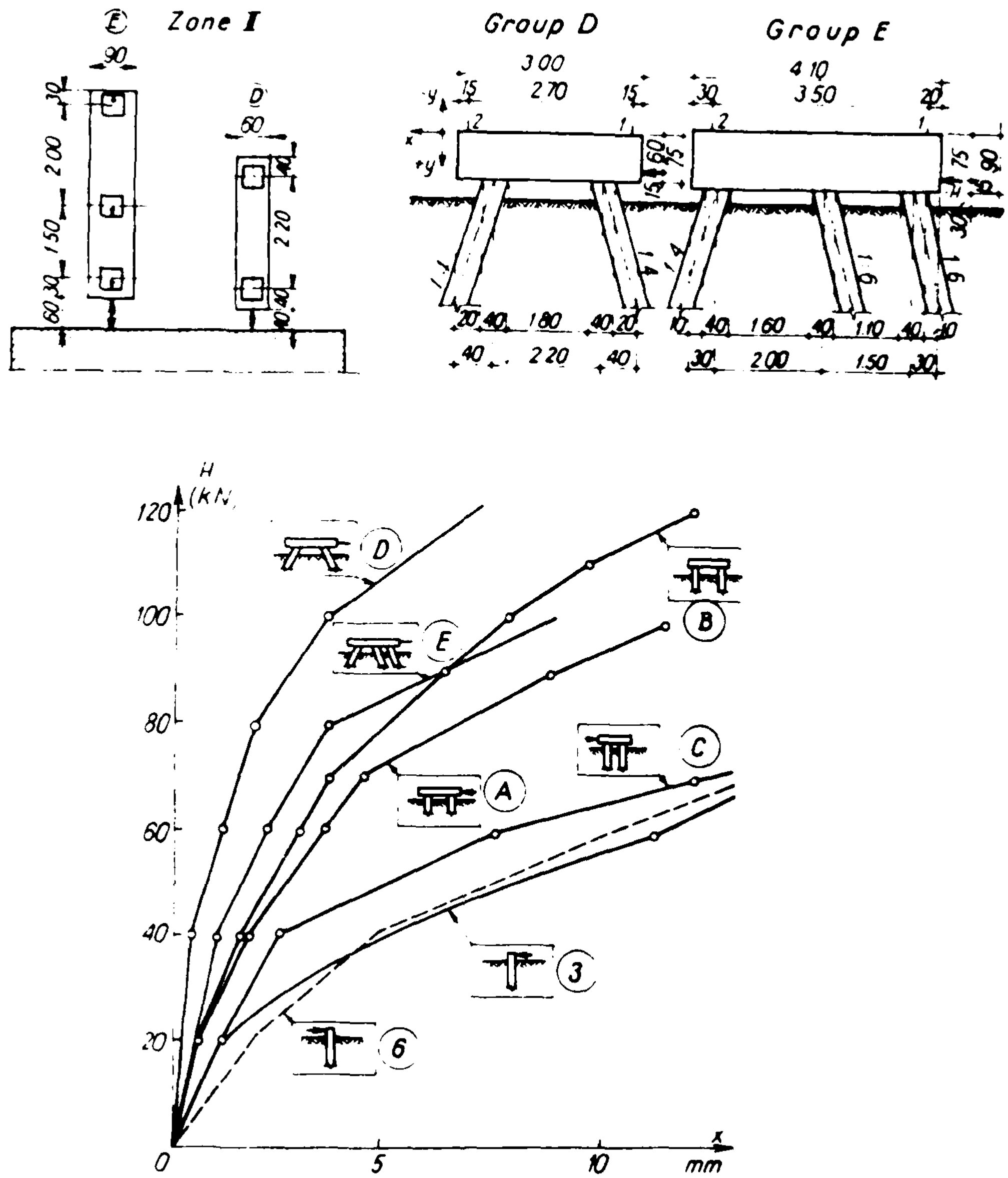


Fig. 5.4 Test Layout for Groups D & E and Lateral Load Test Data - Danube River (after Manoliu et al, 1977)

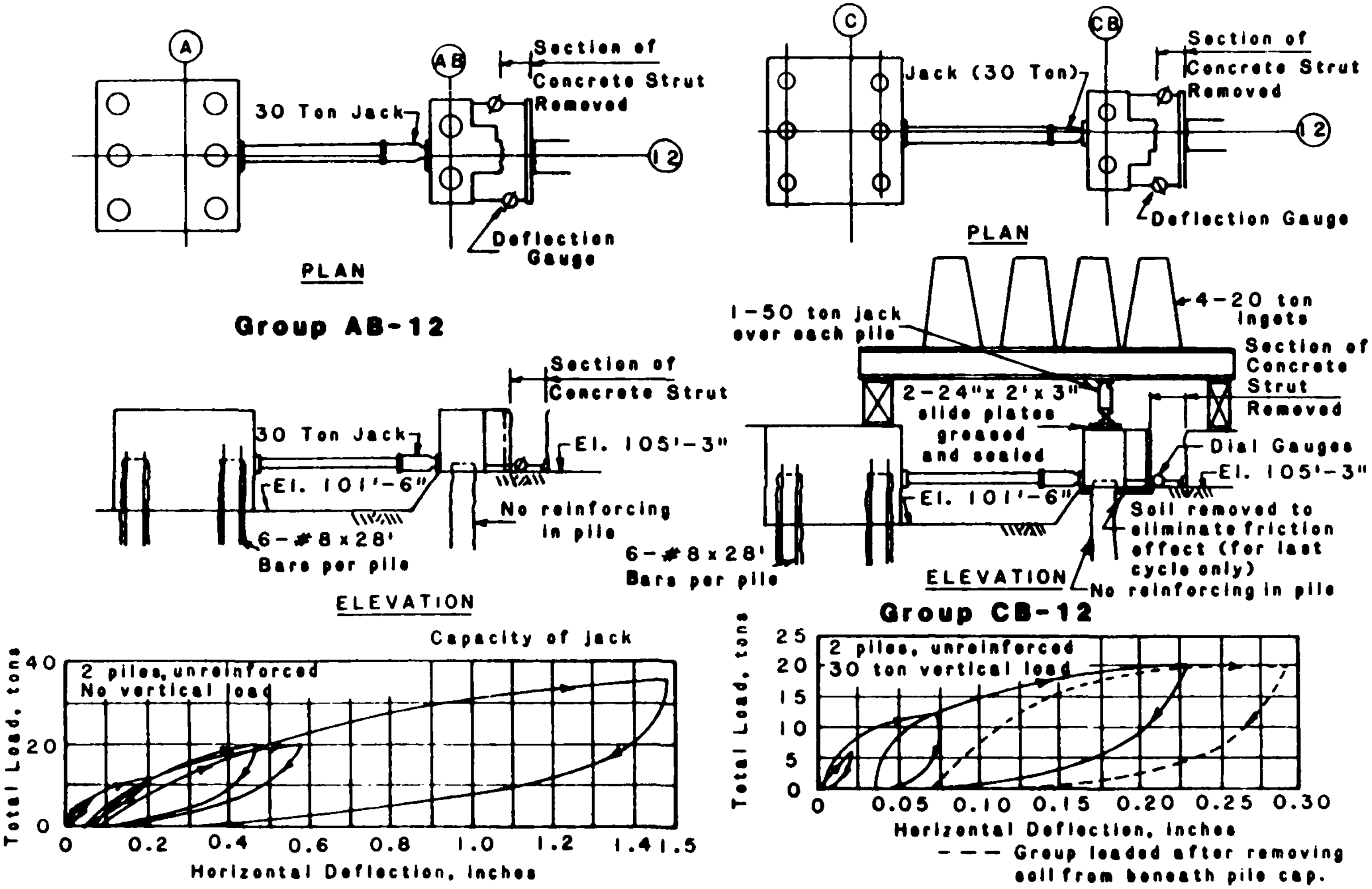


Fig 5.5 Load Test Data - Detroit Steel Mill (after Beatty, 1970)

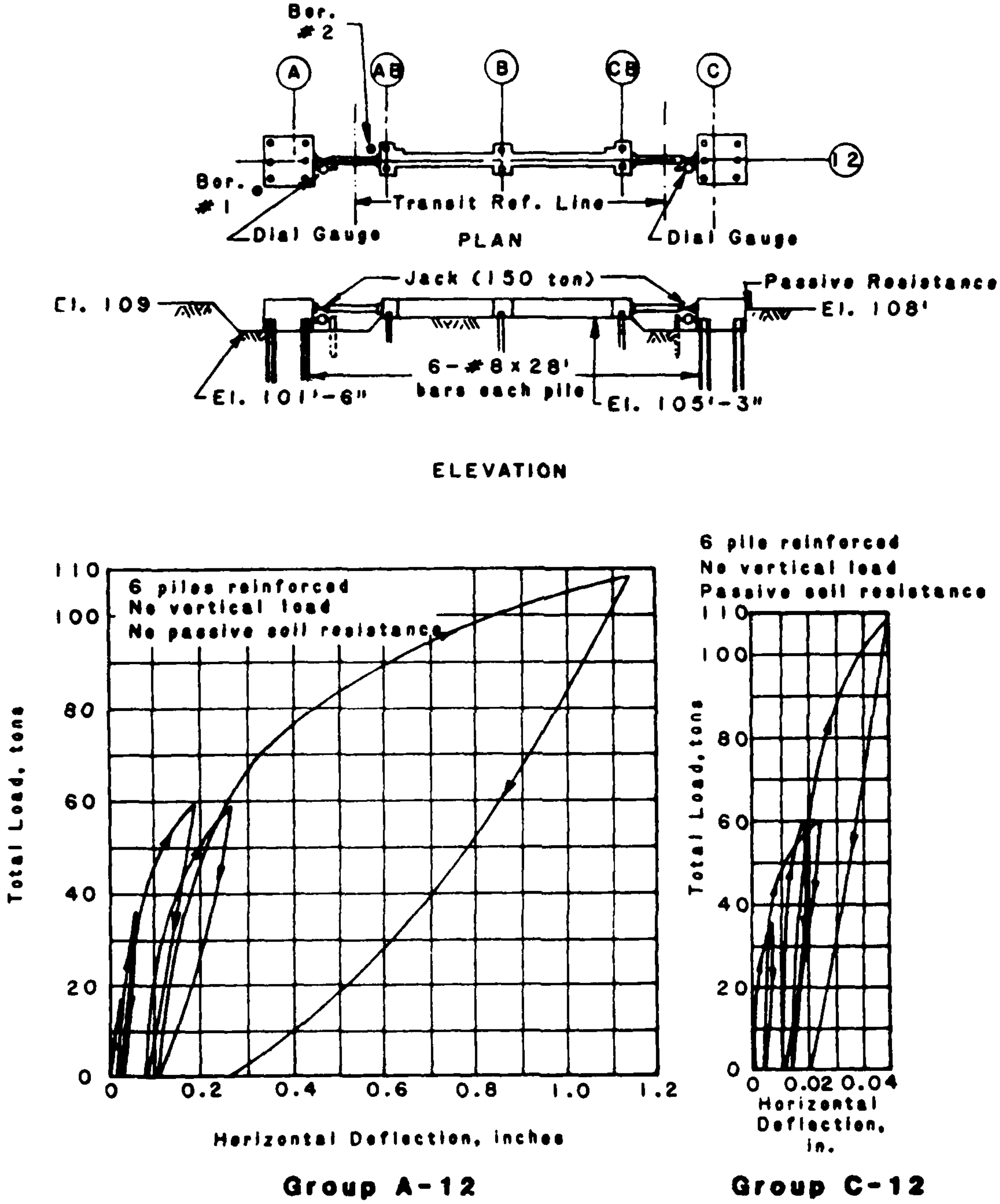


Fig. 5.6 Load Test Data - Detroit Steel Mill

(after Beatty, 1970)

ically applied, although several cycles of load were used. Loads and deflections of the pile caps were measured; load deflection plots of the results of these tests are presented in Figs. 5.5 and 5.6.

Kim and Brungraber - Lewisburg, Pennsylvania

During 1970 and 1971, tests were performed by researchers from Bucknell University (Kim and Brungraber, 1976) on a site in Lewisburg, Pennsylvania. Soils at the site consisted of stiff silty and sandy clays having SPT blowcounts in the range of 30 to 40. The 3 groups of piles each consisted of 6 10BP42 steel H piles spaced approximately 3 ft on-center, and driven to refusal on limestone at a depth of 40 feet. One of the groups contained 4 battered piles. Two isolated piles were also tested. A general layout of the site is presented in Fig. 5.7. All piles were extended into a concrete pile cap, which rested upon but was not embedded into the subgrade. Some piles were instrumented with strain gauges, as shown in Fig. 5.7, to measure bending moments during loading.

Various combinations of vertical and horizontal loads were applied to the piles. Loadings were essentially static and were applied in such a manner as to model the construction and loading of a bridge abutment. Several cycles of some loads were applied, but no series of repetitive loadings were reported. Some loads were held for as long as 24 hours to investigate the effects of short-term, sustained loading.

Some of the results of the Lewisburg study are presented in Figs. 5.8 and 5.9. The loadings that were used did not approach the limiting stresses in the piles, and the load-deflection curves are

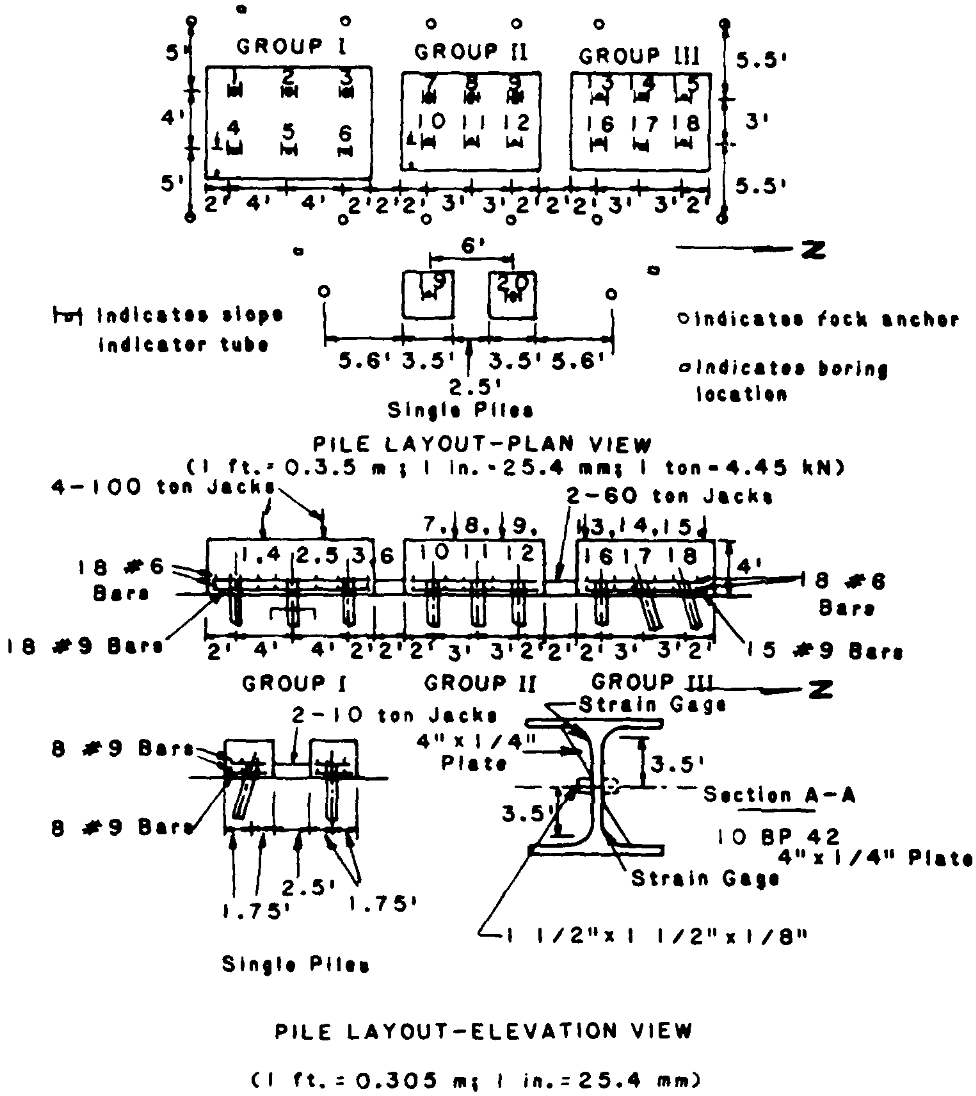
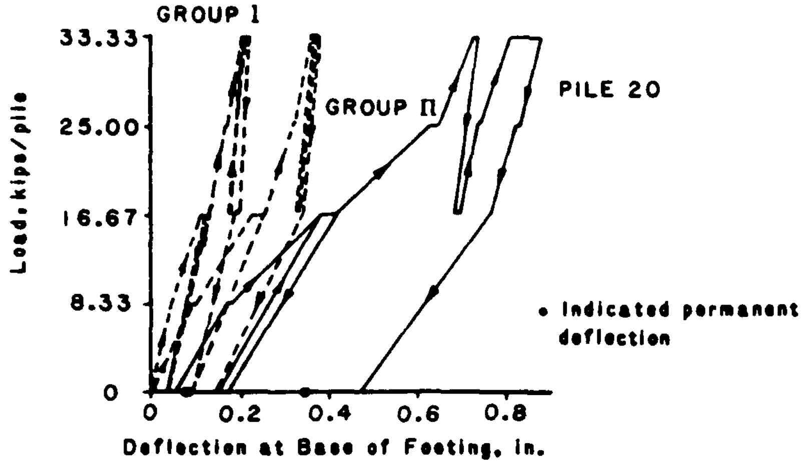


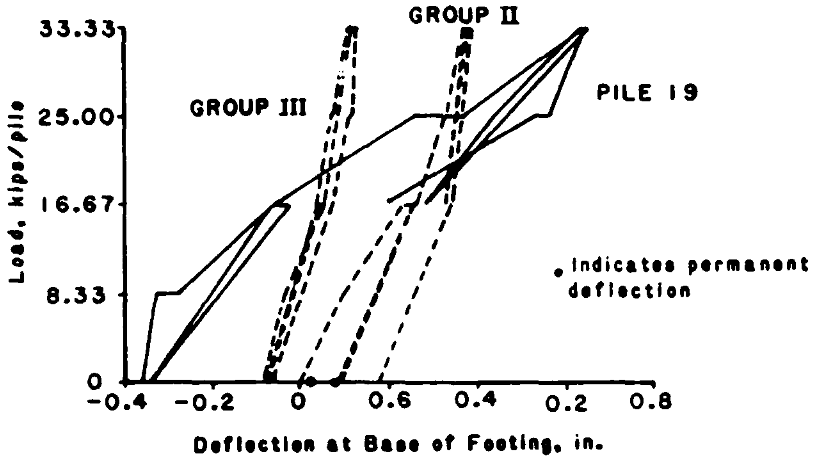
Fig. 5.7 Pile Layout - Lewisburg, Pa.

(after Kim and Brungraber, 1976)



LATERAL DEFLECTIONS–GROUPS I AND II, AND PILE 20

(1 in. 25.4 mm; 1 kip 4.45 kN)



LATERAL DEFLECTIONS–GROUPS II AND III, AND PILE 19

(1 in. 25.4 mm; 1 kip 4.45 kN)

Fig. 5.8 Lateral Deflections – Lewisburg, Pa.

(after Kim and Brungraber, 1976)

AD-A193 498 BEHAVIOR OF A LARGE-SCALE PILE GROUP SUBJECTED TO CYCLIC LATERAL LOADING..(U) TEXAS UNIV AT AUSTIN GEOTECHNICAL ENGINEERING CENTER D A BROWN ET AL. 2/5

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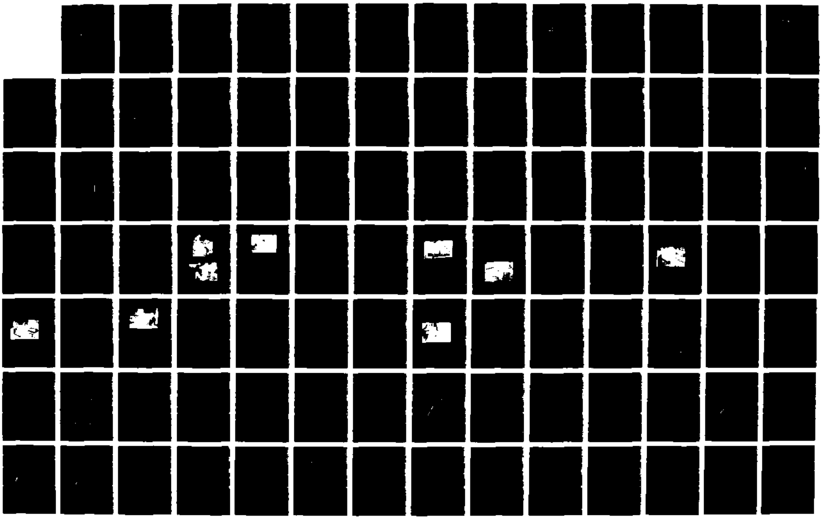

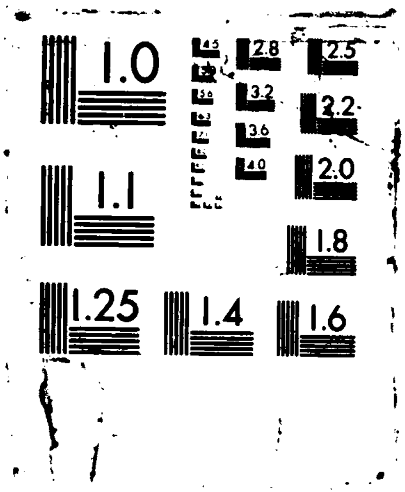
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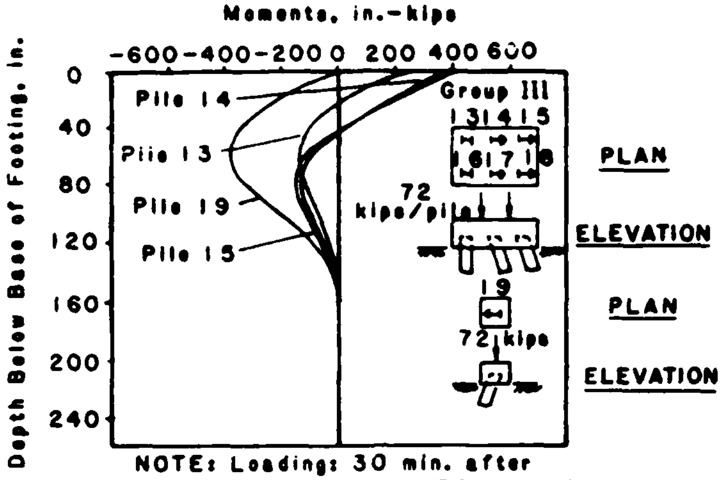
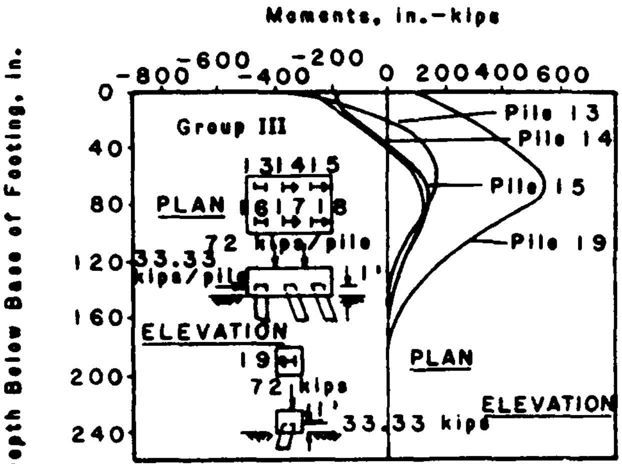


NOTE: Loading: 30 min. after
72 kips/pile (320.4 kN)
Vertical Load, 1st Cycle, Series B

MOMENTS IN PILES-GROUP III AND PILE 19
(1 in. = 25.4 mm; 1 in.-kip = 113 N-m)



NOTE: Loading: 72 kips/pile (320.4 kN)
Vertical Load, 33.33 kips/pile
Lateral Load, 30 min. after
Lateral Loading, 1st Cycle, Series B
33.33 kips 148.3 kN

MOMENTS IN PILES-GROUP III AND PILE 19
(1 in. = 25.4 mm; 1 in.-kip = 113 N-m)

Fig. 5.9 Moment in Piles - Lewisburg, Pa.
(after Kim and Brungraber, 1976)

nearly linear up to the maximum load that was applied. Bending moments shown in the figures indicate the maximum negative moments at the pile cap. These moments have been extrapolated from the nearest gauge level. Kim and Brungraber noted that the results indicate an efficiency of the groups of greater than one, when compared to the single-pile-test results. Matlock (1976) pointed out the fact that differences in top restraint could account for much of the increase in load resistance of the group, but also computed an efficiency of greater than one when taking restraint into account. The frictional resistance of the sliding cap under the substantial vertical load could have added significantly to group resistance.

Other observations included by Kim and Brungraber are as follows.

1. The effects of sustained loading were small.
2. Batter piles driven to bear on rock were substantially more effective than vertical piles in resisting lateral load.
3. Pile groups containing unsymmetrically placed batter piles can develop sizeable lateral displacements and bending stresses under vertical loading only.
4. Increased spacing from 3 to 4 ft between piles resulted in a large increase in the load required to produce a given deflection (note that the group with the 4 ft spacing had a larger pile cap as well).

5. A limited number of cycles resulted in a small increase in lateral deflections, but considerably larger increases in the bending stresses.

Although the piles in the Lewisburg tests were more thoroughly instrumented than those in any previous group test, several criticisms of the test program prevent full utilization of the test data for comparison with available analytical techniques. Some examples are:

1. The pile caps were resting on soil and supported an unknown portion of the load applied to the groups.
2. The strain gauges shown in Fig. 5.8 are located at positions where the strain may not be representative of the bending moment. No mention was made in the 1976 article of calibration of the instrumentation, and the presence of a cover plate and of the web to flange corner would be expected to influence the strain at the gauge location.
3. There can be errors involved in the extrapolation of measurements of bending moment below grade to get negative moments at the pile cap. End effects from the pile cap introduce errors, as does the unknown resistance of the soil to deflection between the topmost gauge location and the base of the cap.
4. Some changes in soil properties could have occurred during the time the tests were in progress; the site was not reported to have been flooded during this period.
5. As mentioned by Matlock and Foo (1976) in their discussion of the tests, the different end conditions of the single

pile and group piles do not allow direct comparison of the results.

6. The soil properties and soil test results that were presented fail to give sufficient detail for a thorough study of predictive methods.

Matlock, Ingram, Kelley, and Bogard - Harvey, Louisiana

Matlock et al (1980) reported the results of field experiments with groups of laterally-loaded piles in a very soft clay at Harvey, Louisiana. These tests were actually performed in the late 1960s under the sponsorship of several oil companies, but was kept proprietary for some time. Six series of tests were made; circular groups of five and ten piles were loaded statically and cyclically, and single piles were also loaded similarly. Cyclic degradation in the response of single piles has been observed experimentally and well-documented. The Matlock et al study represents the first and, in fact, only large-scale experiment prior to this current research in which many cycles of load have been applied to a pile group.

The piles were 6.625 in. diameter by 45-ft-long steel pipes. The piles were jacked into place through a template. The template, shown in Fig. 5.10, also served as the loading frame. Moment-free connections to the piles were made at two points as shown in the figure, and these connections were instrumented to allow measurement of load to many of the piles. Center-to-center spacing of the piles installed through the template was 1.8 diameters for the 10-pile groups and 3.4 diameters for the 5-pile groups. Bending-moment transducers consisted of strain gauges attached to a device that could be

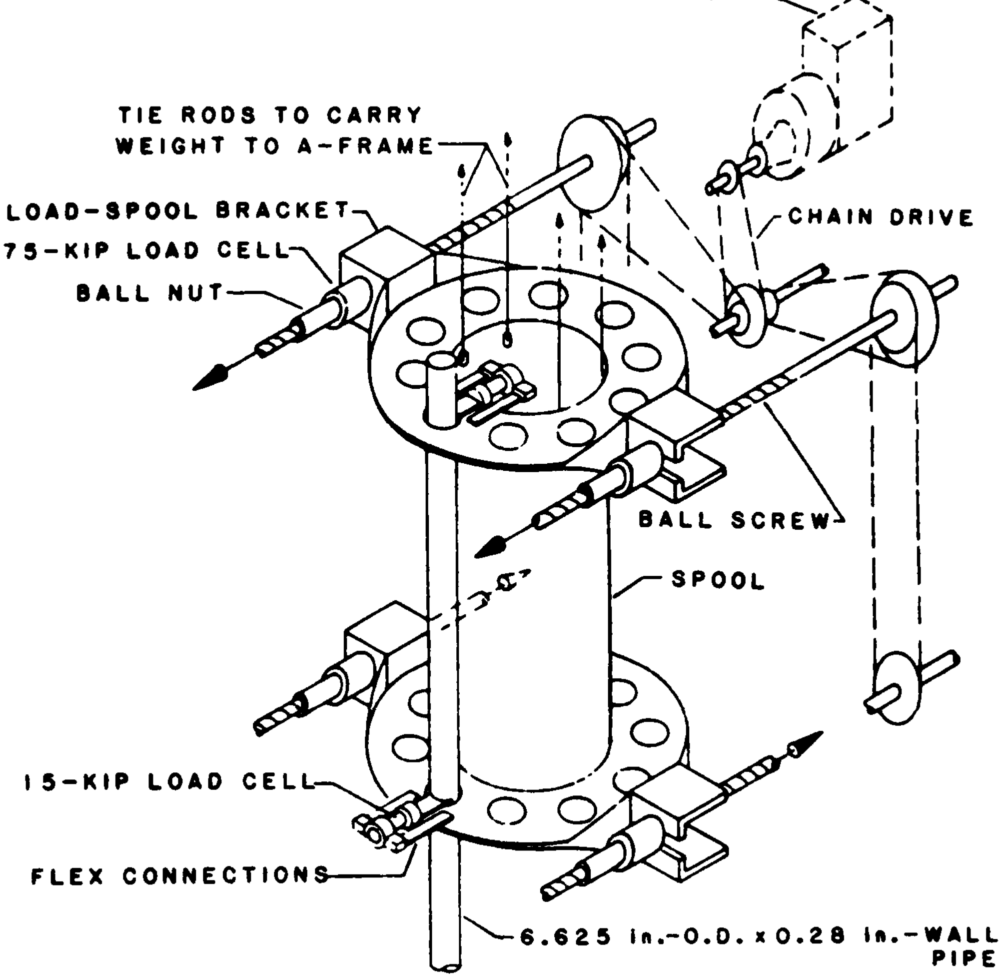


Fig. 5.10 Drive Mechanism and Spool for Loading the Test Piles - Harvey, La. (after Matlock et. al., 1980)

affixed to the inside of a pile after the piles were jacked into place. These transducers were placed within the single piles and within selected piles of each group to allow some indication of the distribution of bending moment with depth. It should be noted that these transducers were not used in sufficient number to allow definitive interpretation of soil resistance and deflection as a function of depth; therefore, p-y curves could not be obtained from the data.

Soils at the site consisted of very soft to soft dark gray organic clay with water contents and liquid limits in excess of 100. Shear strengths were determined by unconfined and UU-type triaxial tests as well as by in-situ vane tests. Undrained shear strengths ranged from 1 to 3 lb/sq inch. The piles were installed into a 15 ft wide by 8 ft deep pit to avoid near-surface peats. The sides of the pit were supported by steel sheet piles embedded 6 ft below the dredgeline.

Loading was controlled by monitoring displacement at several points and applying load to achieve a given displacement. Cyclic loading was performed between predetermined forward and reverse displacements from the original no-load position, with the reverse displacement set at a constant 10% of the forward displacement. At least 100 cycles were applied at each displacement setting. For the static tests and for the cycles which data were to be taken, the load was held until pile reactions were stabilized. Load and moment transducers were then read and recorded. The time required for the load to stabilize was reported to be usually about ten minutes.

Some of the results of the Harvey experiment are summarized in Figs. 5.12 and 5.13. Noteworthy observations and conclusions made by Matlock et al include the following.

1. No clearly distinguishable pattern of variation was found among reactions for the individual piles in a group; variation in bending moment generally was less than 10% from the average of the group.

2. Less resistance per pile was developed at a given deflection for increasing number of piles in the group (at a closer spacing).

3. The onset of cyclic deterioration of resistance in each of the pile groups occurred at approximately the same small threshold deflection.

4. Cyclic deterioration was more severe for the 10-pile group than for the 5-pile group. Cyclic-load response of the 5-pile group tended to converge toward that of the single pile. The gapping of the soil around the piles during cyclic loading was observed to merge into a single gap in the more closely-spaced 10-pile group.

The authors noted the highly inelastic response of the soft clay soil and concluded that the results give little encouragement to current (1980) concepts of elastic interaction among piles in a group.

The most significant criticism of the Harvey study is the fact that the tests were conducted in a relatively deep pit as shown in Fig. 5.11. The effect of this overburden as well as the embedded sheet piles supporting the pit walls is uncertain.

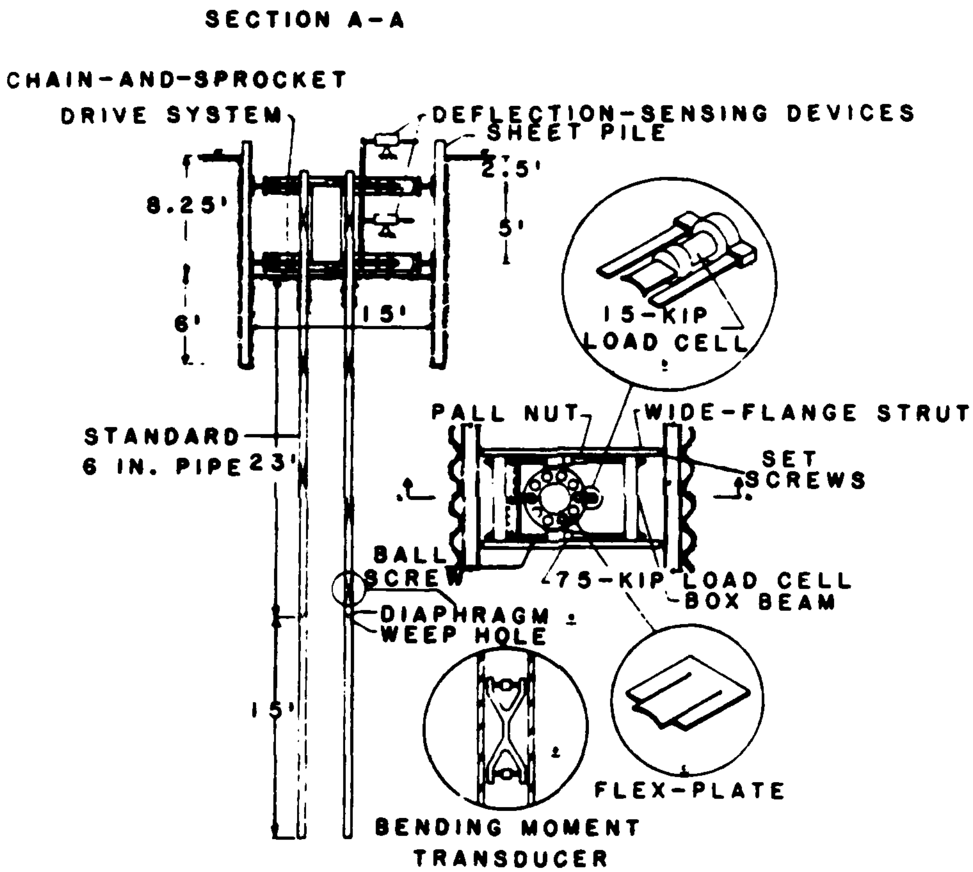


Fig 5.11 Arrangement of Equipment for Field Tests - Harvey, La. (after Matlock et. al., 1980)

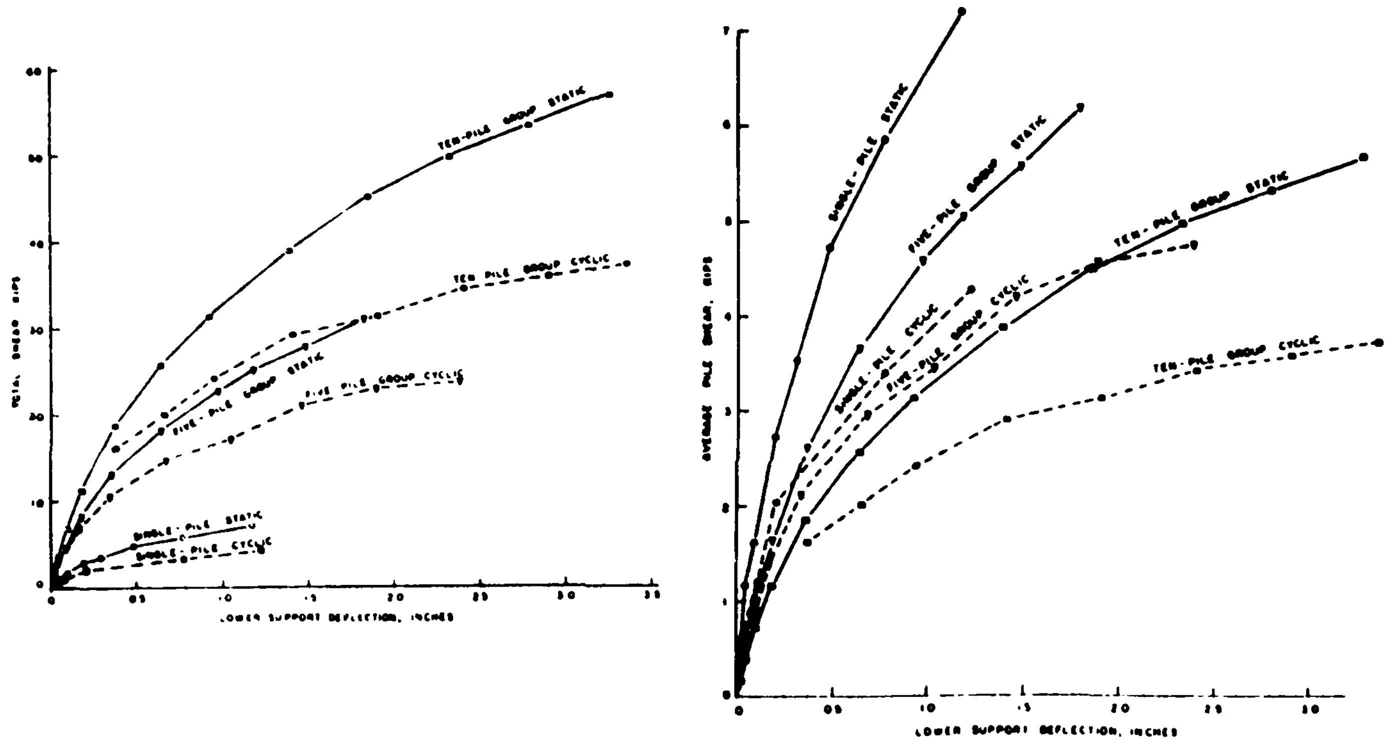


Fig. 5.12 Load vs. Deflection Measurements - Harvey, La.
(after Matlock et. al., 1980)

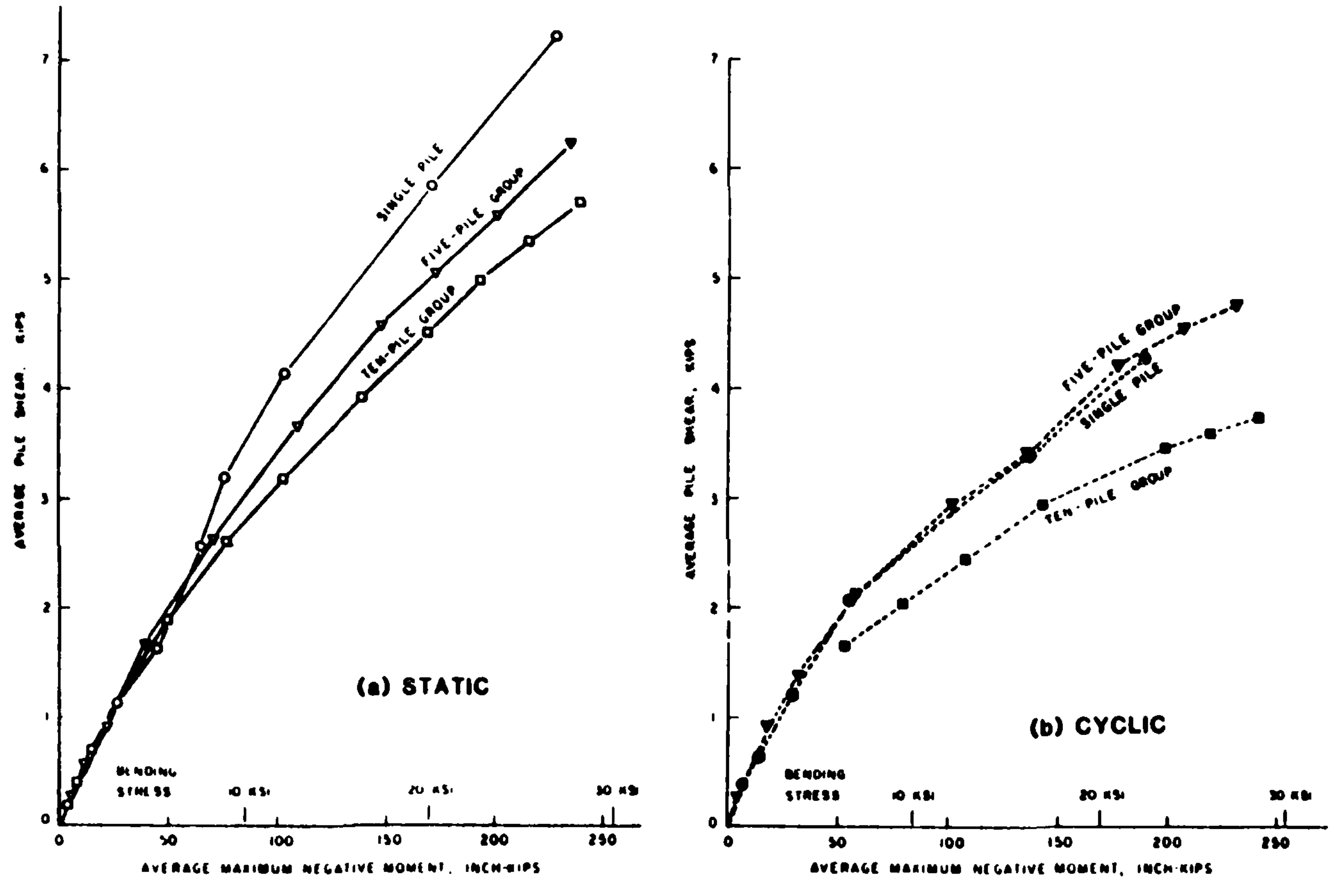


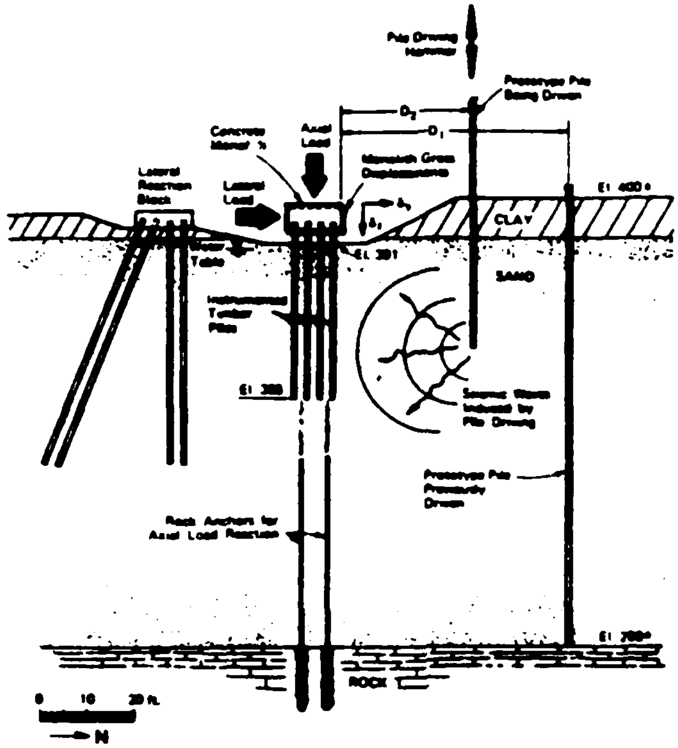
Fig. 5.13 Variation of Maximum Negative Bending Moment with Load - Harvey, La. (after Matlock et. al., 1980)

Holloway, Moriwaki, Finno, and Green - Lock and Dam 26

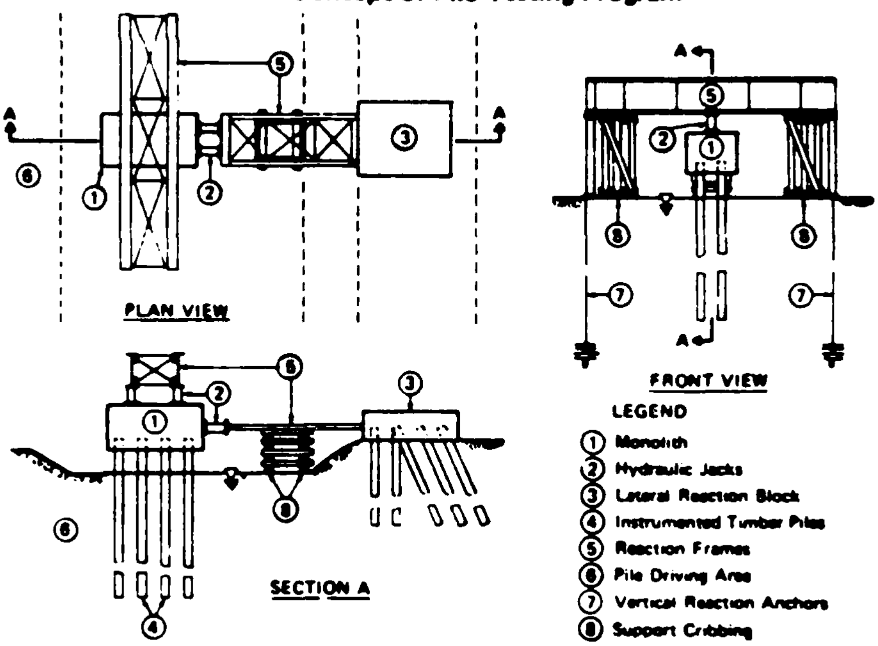
A comprehensive pile-test program was conducted near Alton, Illinois in conjunction with the investigation of rehabilitation of Lock and Dam 26 on the Mississippi River. As part of this program, an 8-pile group of timber piles in sand was loaded laterally to failure under constant axial load (Holloway et al, 1981).

The test program was designed to model the behavior of an existing monolith in the dam as nearly as possible, and was thus subjected to some "conditioning" prior to testing to failure. Timber piles with a butt diameter of 14 in. were installed to a depth of 35 ft on 3 ft centers in a 2 x 4 group. The piles were installed by jetting to a depth of 25 to 30 feet. The driving of the final 5 ft of penetration was done with a Vulcan No. 1 air hammer. The authors reported that in-situ tests on the alluvial sands indicated a marked increase in penetration resistance and levels of horizontal stress in the upper 20 ft after driving, and that the adverse effects of jetting should thus have been more than offset by densification from driving the final 5 ft of the piles. The results of the in-situ tests performed after driving indicated the sand to be dense to very dense.

The piles were embedded into and loaded through a massive concrete pile cap that was not in contact with the ground during the test. The test arrangement is sketched in Fig. 5.14. Figure 5.15 illustrates the instrumentation provided on the test piles, that included strain gauges and telltales for monitoring load transfer. Inclinometer casing on the corner piles allowed the measurement of lateral movements with depth.



Concept of Pile Testing Program



Typical Test Setup

Fig. 5.14 Test Setup - Lock and Dam 26 (after Holloway et. al., 1981)

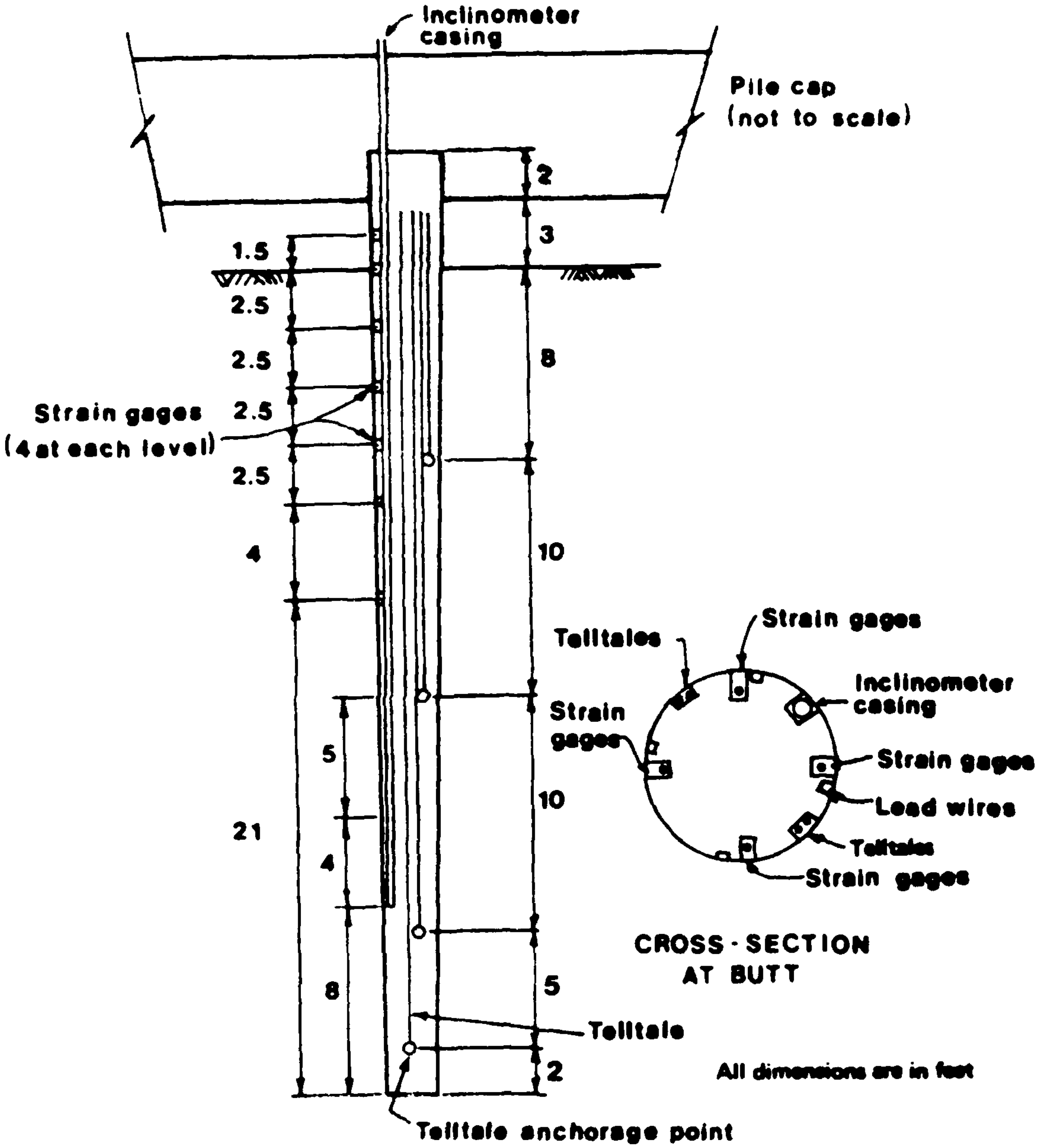
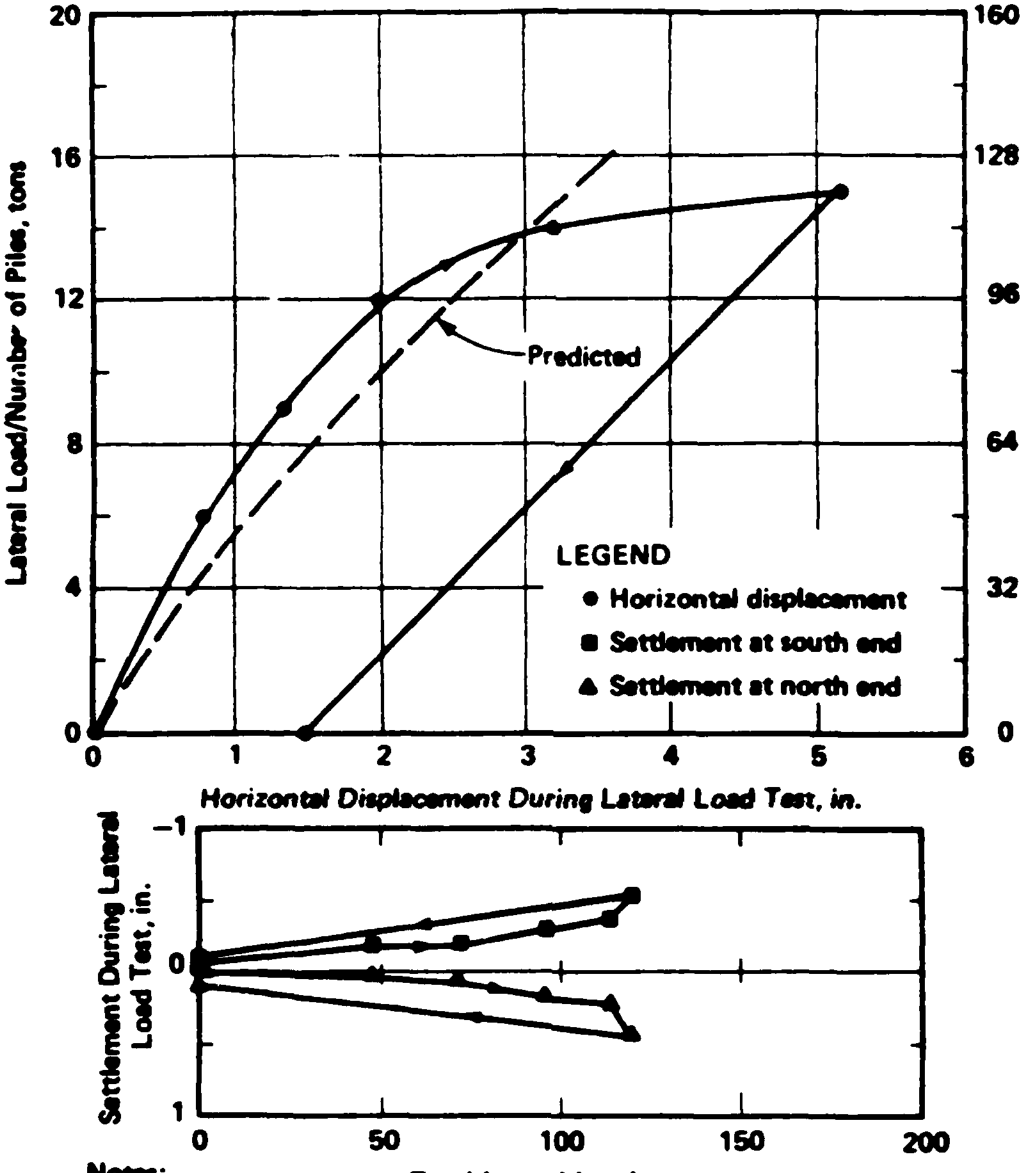


Fig. 5.15 Timber Pile Instrumentation - Lock and Dam 26 (after Holloway et. al., 1981)

Prior to testing the group to failure, the group was subjected to cyclic lateral loading at an intermediate load, and a sustained lateral load while driving steel piles nearby. At the end of these "preconditioning" loads, permanent pile cap displacements of 1 to 1.5 inches in the vertical direction and 2.2 inches in the horizontal direction were measured. The group was then loaded to failure while maintaining a constant vertical load of 240 tons. The load was applied in steps, and data from the instruments were read after the rate of horizontal displacement reached 0.01 in./hr or less. Failure was defined as the load at which the rate of horizontal displacement did not stabilize within a few hours.

Figures 5.16 and 5.17 present some of the results of the experiment, along with predictions made by Holloway using the PGROUP2 (similar to PILGP2R) model described in this report. Holloway et al made several interesting observations concerning the results.

1. Significant differences in response occurred between the front and rear piles.
2. The greatest differences in soil resistance between the front and rear piles occurred at the shallower depth, with the front piles encountering a stiffer and stronger soil resistance.
3. Cyclic-lateral loading softened the soil resistance at shallow depths.
4. Vibrations from driving nearby piles produced significant pile-soil deformations with only minor changes in mobilized soil resistance.



Notes:

1) Axial load during test was 240 tons
2) Horizontal displacement prior to load testing was 2.20 in.
3) Settlement prior to load testing was 0.95 in. at south end and 1.54 in. at north end
4) Residual displacement after complete unloading (axial and lateral)

Fig. 5.16 Pile Cap Displacements During Lateral Load Test - Lock and Dam 26 (after Holloway et. al., 1981)

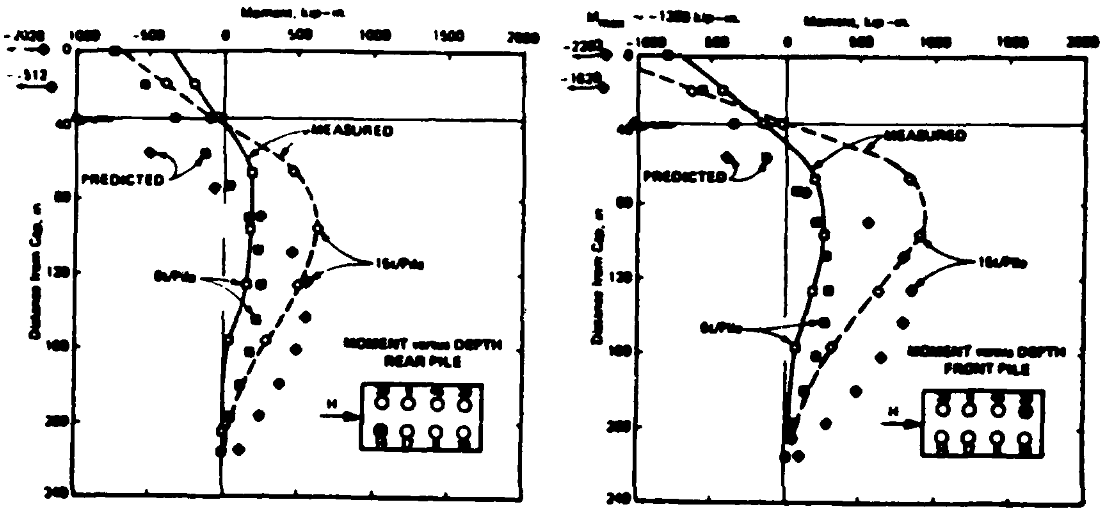
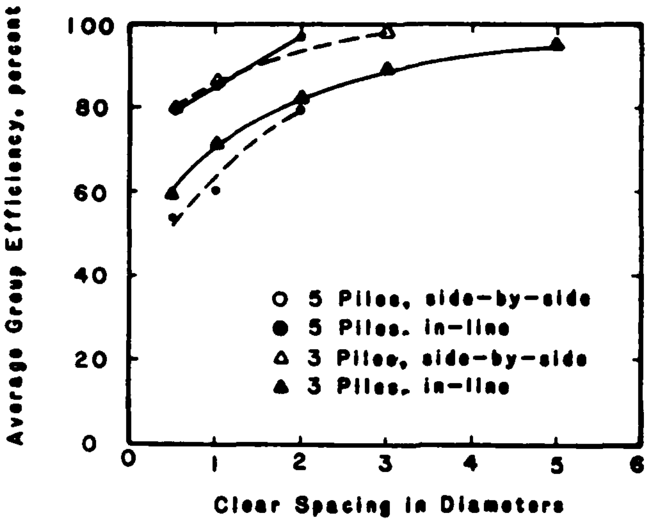


Fig. 5.17 Comparison of Measured and Predicted Moment Distributions - Lock and Dam 26 (after Holloway et. al., 1981)

Cox, Dixon, and Murphy - Amoco Model Piles

A series of model tests of short, 1-in. diameter piles were conducted to investigate the efficiencies of pile groups (Cox, Dixon, and Murphy, 1984). In a parametric study, tests were performed on groups of 1, 3, and 5 piles arranged in in-line and side-by-side configurations. Penetrations of 2, 4, 6, and 8 diameters were used on clear spacings of 0.5, 1, 2, 3, and 5 diameters. The tests were performed in a very soft clay, placed within a 25 in. square by 16 in. deep box. The soil was a Wilcox clay of moderate plasticity that was placed at a water content very near the liquid limit (61%). Details were not given on the method of compaction, but miniature vane tests run in situ indicated an undrained shear strength of about 42 lb/sq foot.

The lateral loading was applied to the piles in one direction at a slow rate of movement. The piles were rigidly held in a loading frame and loaded with a constant deflection for all of the piles in the group. Instrumentation provided for measuring load and displacement in each of the piles, as the principal objective was to determine the efficiency of various configurations with respect to the single piles. The results of the study are summarized in Fig. 5.18 as a plot of average group efficiency vs. clear spacing, expressed in pile diameters. The term "efficiency" is defined as the load on a pile in a group divided by the load on a single isolated pile at the same deflection. As one might expect, the in-line piles had much lower efficiencies than the side-by-side groups, with decreasing efficiency with decreasing spacing.



NOTES:

1. Efficiencies expressed as a percentage of an equal number of isolated piles.

2. Efficiencies are averages of all penetrations of a given test type.

Fig. 5.18 Efficiency of Pile Groups - Model Piles (after Cox et. al., 1984)

SUMMARY AND CONCLUSIONS

A number of experiments have been described. Most of the experiments were performed to investigate a specific situation and were not designed to be general. Three of the experiments with groups (Lewisburg, Harvey, and Lock and Dam 26) have had some provision for measuring bending moments with depth. Of these three, only the Harvey and the Lock and Dam 26 studies had provisions for determination of the distribution of load to the piles. Only the Harvey experiment incorporated repetitive cyclic loading to failure under conditions approaching offshore loadings. The Harvey experiment was performed on 6.625-in. piles in a very soft clay, while the Lock and Dam 26 experiment was performed in sand.

In considering the objectives of the current research, it appears that only the Harvey experiment performed by Matlock et al offers a basis for comparison. Even though the Harvey tests were similar to the current research in some ways, that study was performed in much softer soil with more limited instrumentation (i.e., fewer strain-gauge installations). It appears that the current research program, involving a cyclically loaded and well-instrumented group in stiff clays, will provide much needed information not previously available. In view of the extremely limited test data that are available, it could also be concluded that <u>any</u> well-instrumented test on a pile group would provide much needed information.

CHAPTER 6

TESTING PROGRAM

INTRODUCTION

The primary objective of the testing program was to conduct a well-instrumented test of a pile group under lateral loading to obtain data that will aid in explaining how pile groups behave during lateral loading and what factors influence this behavior. In order to make the most of the research funds that were available, the testing program was designed to utilize an existing group of piles at a site on the campus of the University of Houston that had already been the subject of an extensive geotechnical investigation. The near-surface soils at the site consist of stiff, overconsolidated clays of the Beaumont formation, a Pleistocene-aged, terrace deposit. The group of piles was the subject of an FHWA-sponsored research project concerning an investigation of the behavior of groups of piles under axial loading. The characterization of the site and the behavior of the piles during and after installation was well documented and reported in several excellent articles in journals (Mahar and O'Neill, 1983; and O'Neill, Hawkins, and Audibert, 1982). For purposes of the current study, the site was flooded for an extended period in an attempt to saturate the soil. Water was kept above the ground surface and, when testing the piles, cyclic loading was used to simulate typical loadings from storms in an offshore or waterfront environment. Details of

the arrangement for the test and procedure that was employed are presented in the paragraphs that follow.

INSTALLATION AND HISTORY OF THE PILES

Details of the installation of the piles and the arrangement for testing them under axial loading are well documented by O'Neill et al (1981). A brief summary is presented below.

The piles used for the O'Neill study consisted of a group of 9 steel pipes, 10.75 inches in outside diameter, with wall thicknesses of 0.365 inches. The piles were installed in 1979, in a 3 by 3 arrangement with a nominal spacing of 3-pile diameters on centers. The piles were driven closed-ended to a depth of 43 ft (about 40 ft below final grade). Prior to pile driving, a pilot hole was excavated to serve as a guide. This hole was 8 inches in diameter and extended to a depth of 10 feet. Instrumentation revealed that there was little resistance to driving in the upper 21 ft, but the same near-surface soils provided substantial load transfer during subsequent axial-load testing. Apparently, the soils of the upper 20 ft rapidly built up lateral pressure against the wall of the pile because of swelling of the soil after completion of the driving of the piles. Four days after pile driving, lateral earth pressures were measured below a depth of 9 ft and indicated a lateral-earth-pressure coefficient of 2 to 3; this was somewhat higher than the measured in-situ pressures, although the difference was less distinct nearer the surface. The maximum heave of the ground surface was approximately 1 in. near the outer perimeter of the group. The total surface heave that was

observed accounted for about 30% of the theoretical volume of the displaced soil. Results from tests with the static cone, conducted before and 5 months after installation of the group, showed no evidence of a reduction in shear strength. Inclinometer surveys after driving revealed only very slight batter in the piles, averaging about 1.5%. The largest drift occurred in the northeastern-most pile (labelled pile "A" in the current study, see Fig. 6.9) that was battered to the northeast about 3.5%.

The piles of the group were framed together with a large concrete cap and loaded axially to failure several times during 1979 and 1980. Between 1980 and 1982, a vibrator was mounted atop the cap and the dynamic response of the group was measured. The maximum lateral movements that were observed during dynamic testing were about 0.10 inches. In March of 1983, the group was used as a lateral reaction for another load test which caused a deflection of 0.20 in. to the south (see Fig. 6.9 for directions). In the fall of 1983, the current research was begun. The cap was removed and a 2-ft-deep pit was excavated and flooded in October, 1983. The soil around the group remained submerged through the testing program that began in May, 1984.

SOIL CONDITIONS AT THE TEST SITE

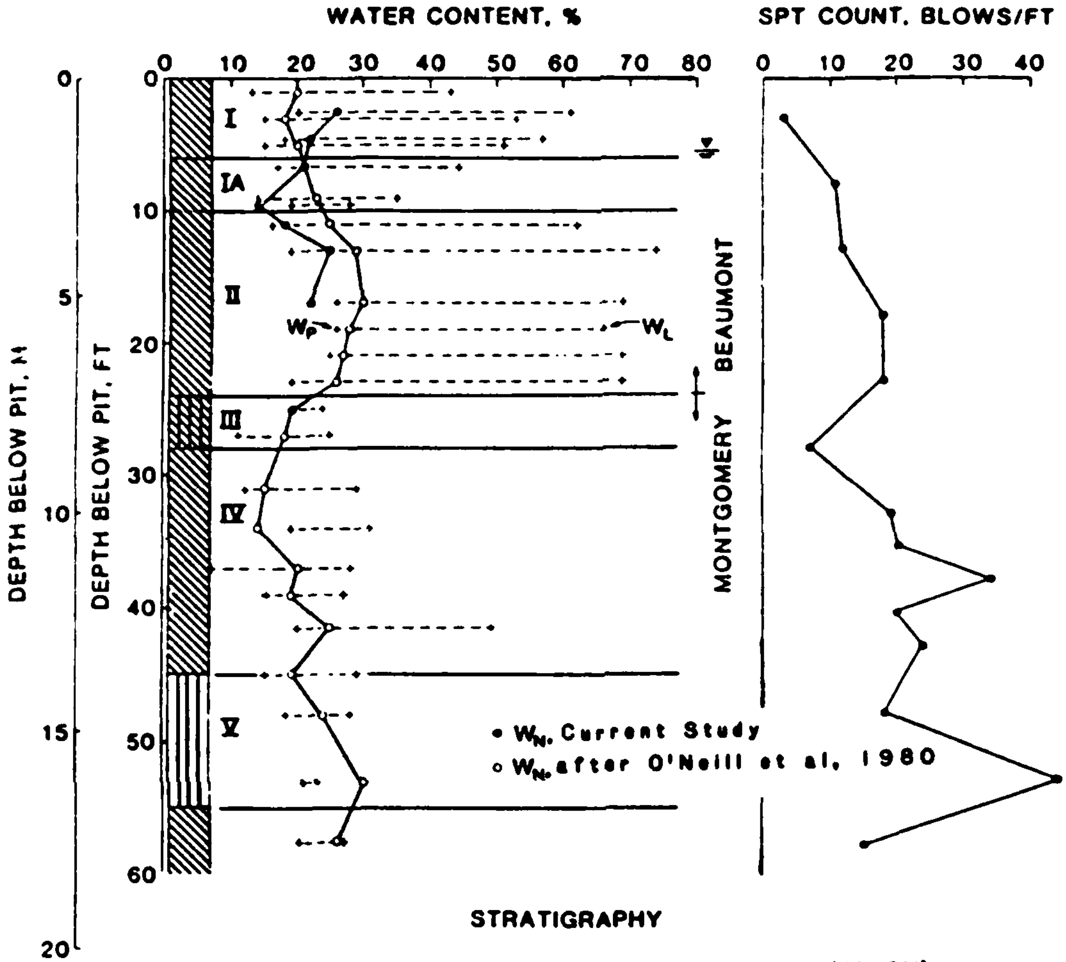
General

No additional investigation of soil conditions has been performed specifically for this study. Work on site investigation was performed for previous FHWA-sponsored research and is presented in

detail in Appendix C of the referenced report by O'Neill, Hawkins, and Mahar (1980). The results of the geotechnical investigatio.. for that study are also presented in a referenced paper by Mahar and O'Neill (1983). Also, considerable additional work has been done in conjunction with proprietary research on the site, and the sponsors of that research have consented to release data on soil conditions for publication in this report. The paragraphs which follow utilize all of this available information to provide a brief description of the soil conditions at the site.

Geology and Stratigraphy

As mentioned previously, the important geological formation for purposes of this research is the Beaumont formation of Pleistocene Age. The stiff, preconsolidated clays and silty clays of this formation extend to a depth of about 24 ft below final grade, thus encompassing the zone of primary importance during lateral loading. Underlying the Beaumont is the Montgomery formation, a similar but slightly older deposit. Both of these formations are deltaic Pleistocene terraces, deposited during interglacial periods and preconsolidated by desiccation during periods of glaciation (when the sea level was lowered). Figure 6.1 illustrates the general stratigraphy of the test site, along with the results of some general classification tests. Geotechnical work revealed the slickensided nature of stratum II, and excavations made after the current group test revealed stratum I to have a secondary structure. The clays of the upper 6 to 8 ft were observed to contain a pattern of very closely spaced joints or



STRATIGRAPHY

I STIFF TO VERY STIFF GRAY AND TAN CLAY (CL-CH)

IA STIFF TO VERY STIFF GRAY AND TAN SANDY CLAY (CL)

II STIFF TO VERY STIFF RED AND LIGHT GRAY CLAY (CH)

III MEDIUM STIFF LIGHT GRAY VERY SILTY CLAY (CL)

IV STIFF TO HARD LIGHT GRAY AND TAN SANDY CLAY (CL)

V DENSE RED AND LIGHT GRAY SILT WITH CLAYEY SILT AND SAND LAYERS (ML)

Fig. 6.1 Stratigraphy at Test Site (1 ft equals 0.305 m)

fissures (on the order of 1/4 in. spacing), making the clay seem friable and crumbly.

Shear Strength

Testing for evaluation of shear-strength parameters has concentrated on undrained laboratory tests and in-situ tests. Such tests have included:

1. unconsolidated, undrained (UU) triaxial tests,
2. isotropically consolidated, undrained ($\overline{\text{CIU}}$) triaxial tests,
3. quasi-static cone penetration tests (CP), using an electronic cone,
4. field, vane-shear tests (FVT),
5. self-boring pressuremeter tests (SBP),
6. K_o consolidated triaxial tests, and
7. consolidation tests.

Items 5, 6, and 7 were performed only during the earlier FHWA work, and thus were not performed after subjecting the site to flooding. The UU tests were performed with a cell pressure of 1.5 times the total overburden pressure, and the $\overline{\text{CIU}}$ tests were performed with consolidation to 1.2 times the effective vertical stress (σ_{vc}). The undrained shear strengths, S_u, from the cone penetration tests were computed from the cone tip resistance, q_c, using:

$$S_u = \frac{q_c - \sigma_v}{N_c}$$

where N_c is the cone tip bearing capacity factor and σ_v is the total vertical overburden stress.

Data gathered from the FHWA study (O'Neill et al, 1980) are presented in Fig. 6.2. These tests were made on the site in its natural condition, before flooding or pile installation. The static-cone-penetrometer values shown in that figure are averages based on three soundings made prior to pile installation (in 1979). These values are based on an assumed value of $N_c = 10$, as presented in the FHWA report. Because that study investigated axial pile capacity, the testing tended to concentrate on soils at fairly substantial depths. Subsequent work has emphasized strengths in the upper 15 ft, as shown in Fig. 6.3. The strengths shown in Fig. 6.3 were determined on the soils below the large 1.5-ft-deep pit, after several months of submergence. Measurements of compression-wave velocity from cross-hole seismic tests indicated that a present saturation in excess of 99% was achieved by this soaking. Only the uppermost few inches became noticeably softened by soaking; pore-pressure changes in the clays below the top few inches produced only subtle changes in soil behavior. Because the N_c value of 10 used in the earlier work was felt to be too low, a value of $N_c = 13.6$ was used in determining shear strength from CPT tests in Fig. 6.3. This value is based upon correlations of cone tip resistance (neglecting the occasional "spikes" in the continuous record of cone tip resistance vs. depth) with vane shear strength.

As is typical of a desiccated clay, these plots reveal considerable scatter in shear-strength data, even for a single type of test. Mahar and O'Neill (1983) hypothesize that the cracks produced during desiccation allow spatially-variable suction pressures in the pore

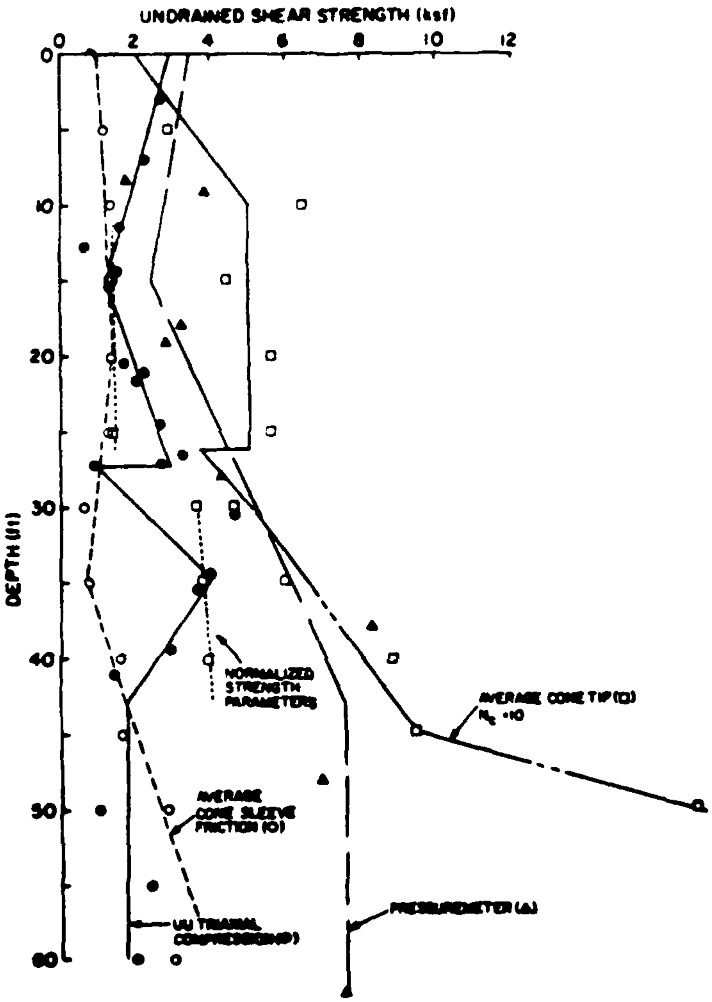


Fig. 6.2 Indicated Shear Strengths
(after O'Neill et al, 1980)

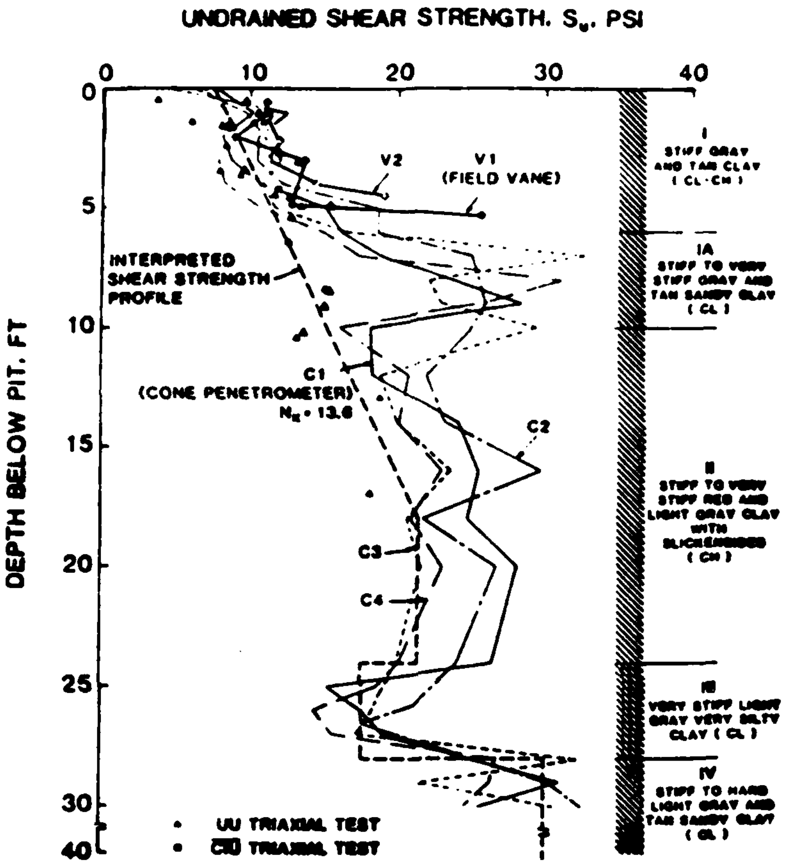


Fig. 6.3 Indicated Shear Strengths

spaces, thus leading to pointwise and directional variability in shear strength and water content. The very close spacing between cracks in stratum I could partially account for the somewhat less prominent variability in this zone. The numerous sand pockets and lenses in stratum IA likely produced much scatter in this zone, especially in CPT values.

The in-situ vane-shear tests readily offer some indication of the sensitivity of the soil. Presented in Fig. 6.4 is a plot of peak and residual shear strengths vs. depth for the upper 5.5 ft below the pit, as indicated by the field-vane tests. Residual values were taken as that indicated after 2 complete (360°) turns of the vane. These data appear to indicate a sensitivity in this zone of about 2.

Other Properties

The existing state of stress in the ground and resulting stress anisotropy affects the interpretation of any measure of undrained shear strength as well as the correlation of S_u to mobilized soil resistances. Determinations of earth pressure at rest (K_o) were made by several methods as a part of the earlier FHWA study and are presented vs. depth in Fig. 6.5. Although no tests were performed on soils shallower than 9 ft below grade (7.5 ft below pit bottom), the trend revealed by Fig. 6.5 implies that K_o is likely to be significantly higher than one in the soils of importance to the behavior of piles under lateral load.

Other material properties which may be important to the behavior of piles under lateral load include the unit weight of the soil and stress-strain characteristics. A plot of wet and dry unit weight

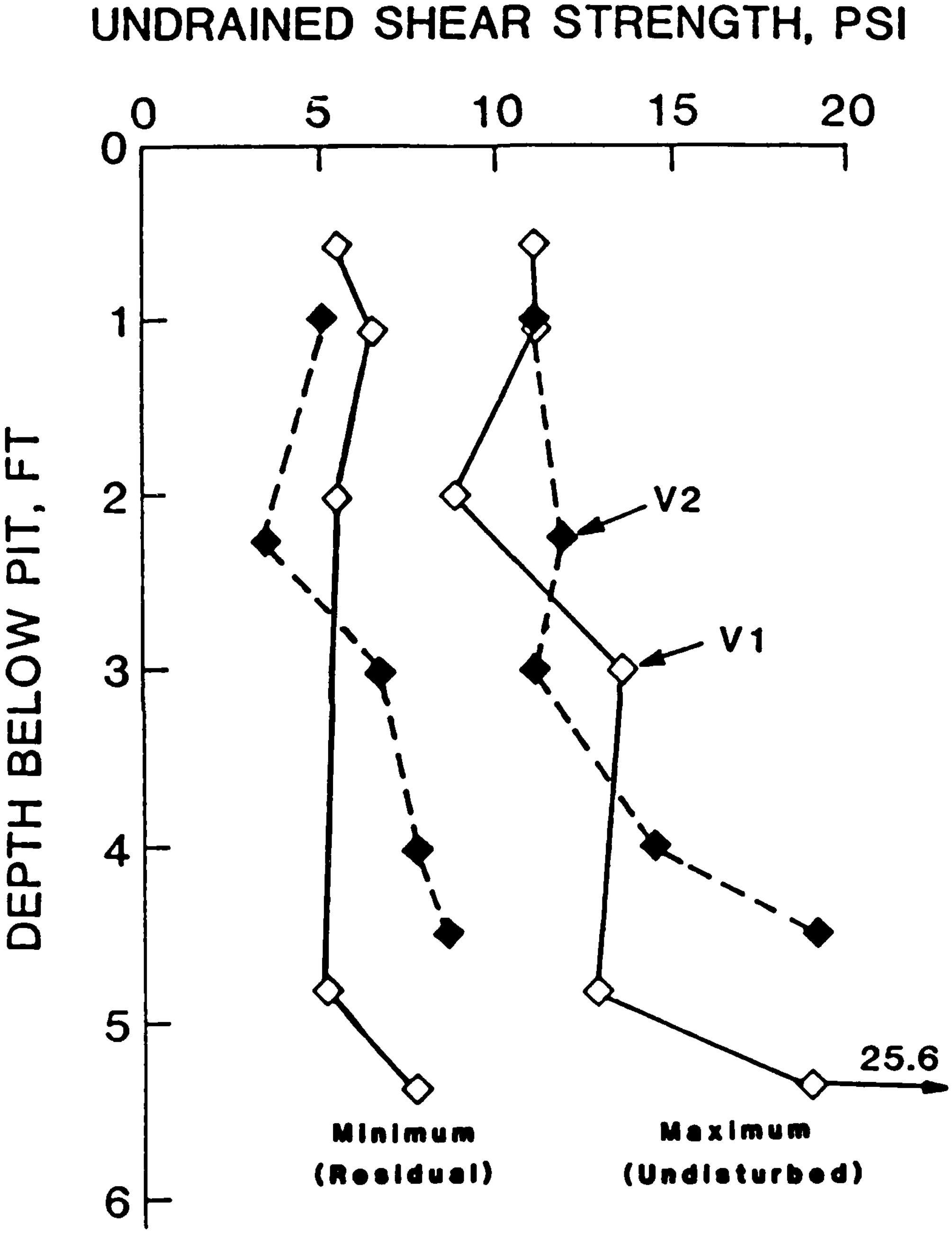


Fig. 6.4 Maximum and Minimum Shear Strength vs. Depth from Vane Shear Tests

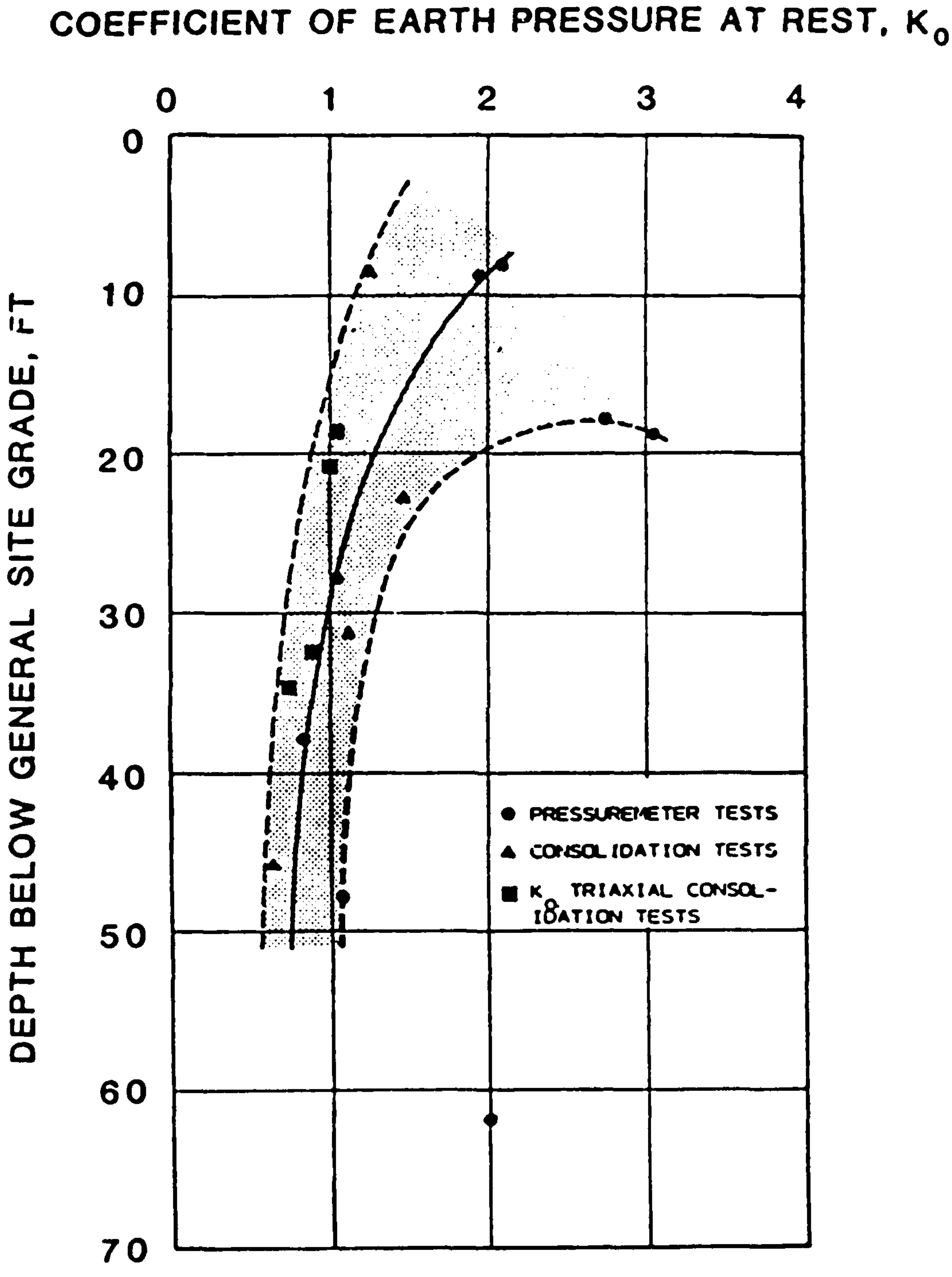


Fig. 6.5 Ko vs. Depth

vs. depth as determined from the triaxial specimens is presented in Fig. 6.6. Actual stress-strain plots from some of the triaxial tests can be found in Appendix C of the report by O'Neill et al (1980). A convenient measure of the stress-strain behavior in a triaxial test is the strain at which 50% of the failure load is achieved, ε_{50}. A plot of ε_{50} vs. depth is provided in Fig. 6.7. Two fairly large ε_{50} values are shown in the upper 1 ft; these apparently reflect a significant softening due to flooding the site.

Summary

Considerable data from a variety of test methods have been presented. Soils of importance to the behavior of piles under lateral load are components of the stiff Beaumont Clay, which are overconsolidated and fissured by desiccation. Material properties vary considerably from point to point, probably due to the nature of overconsolidation by desiccation. Determinations of K_o indicate the major principal stress to be horizontal.

ARRANGEMENT FOR LATERAL TESTING

The testing arrangement for this research was designed after careful consideration of previous research, existing conditions, available funds, and research objectives. The general strategy for the test setup was as follows.

1. In order to simulate submarine conditions as nearly as possible, the site would be flooded for an extended period of time prior to the test. The flooded condition would allow observation of the effects of scour (i.e., the

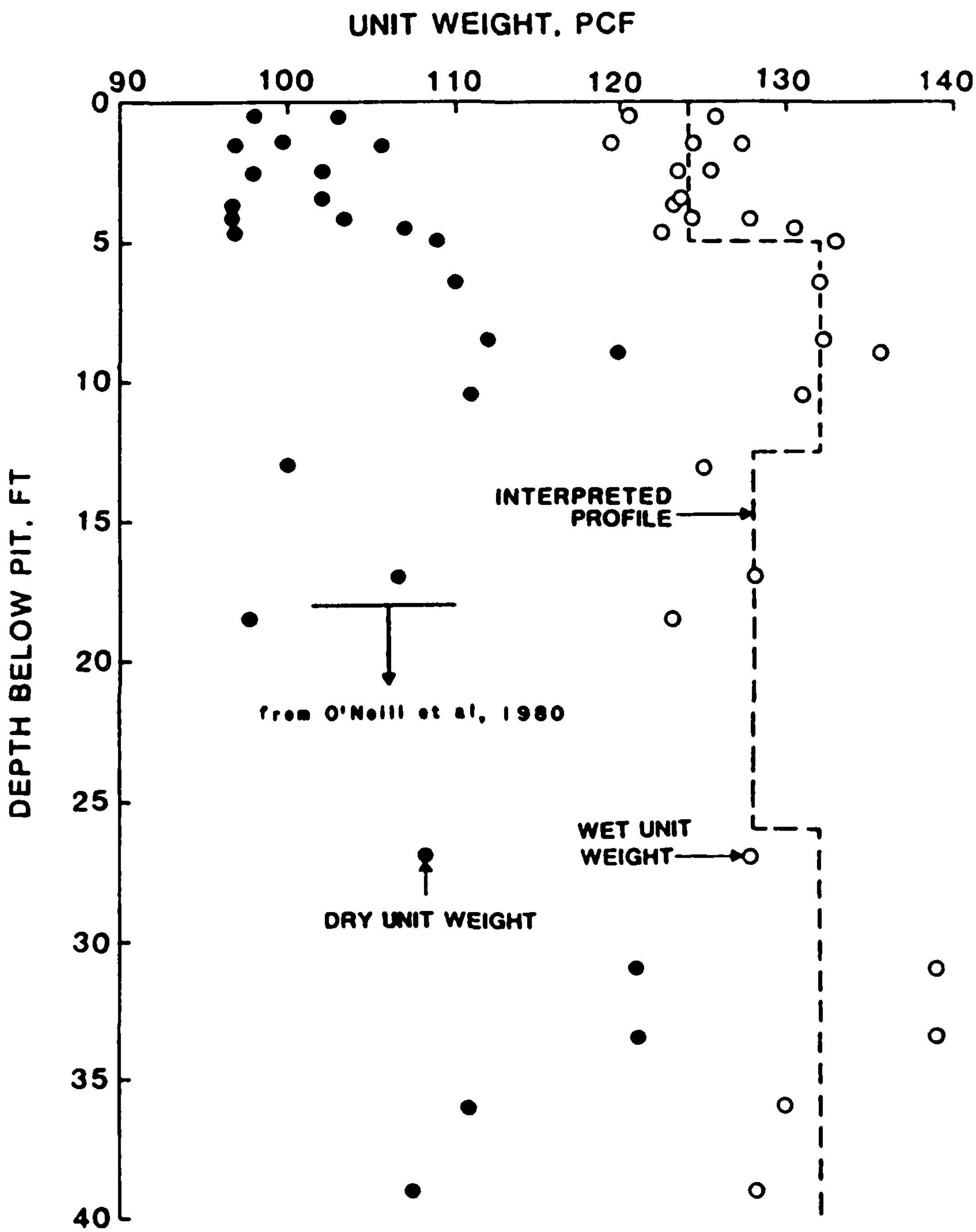


Fig. 6.6 Unit Weight vs. Depth

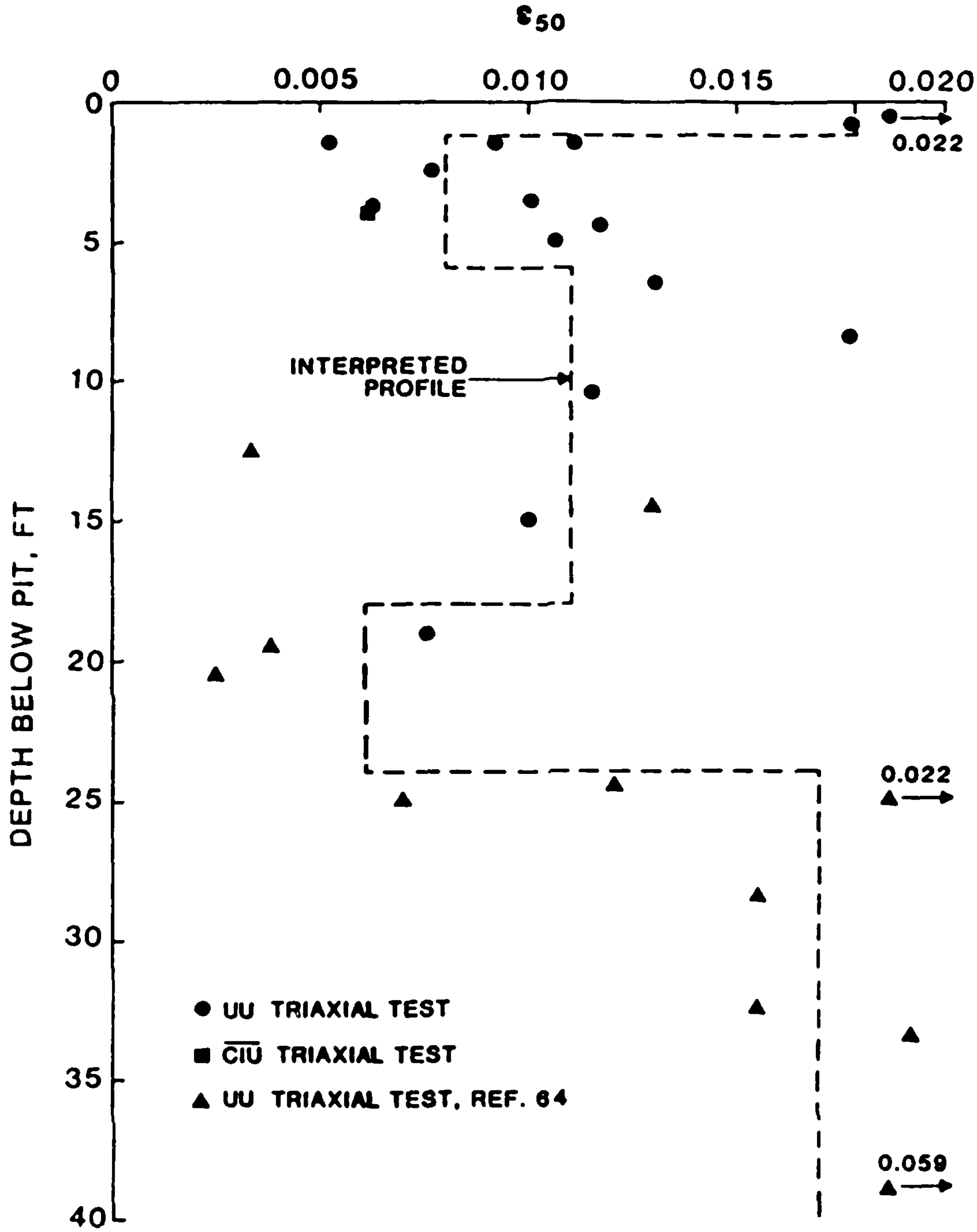


Fig. 6.7 ε_{50} vs. Depth

scouring action of water forced in and out of gaps in the soil around the piles) during the test. The extended soaking period would bring the water content in the near surface clays to a condition at or very near saturation.

2. The bending moment as a function of depth would be measured in all piles. The technique of using strain-gauge measurements to compute pile stresses, bending moments, and p-y curves is well established. Because the piles were already in place, a plan was devised to instrument a smaller diameter pipe which would then be grouted into place in each pile to act as one with the pile. Instrumentation of all nine piles would be desirable, so that actual variations within the group could be measured.

3. The cap would be removed and a loading frame installed, with moment-free connections to each pile. This strategy was dictated by the following considerations.

   a. The piles were not the type of thick-walled section one would desire for a lateral-load test. The actual load that could be transmitted to the soil was therefore somewhat limited due to structural limitations of the piles; large bending moments would occur at the base of the cap. Accurate measurement of loads to individual piles and negative moments at the base of the cap would require a pair of strain-gauge installations above the mudline, separated from the base of the cap by at least 2 ft to avoid end effects on the

instrumentation. A separation of more than 1 ft between these 2 gauge installations would be needed to allow accurate projection of moments to the base of the cap, as well as to allow the shear load to be determined. The 3 to 4 ft or more needed above the mudline for instrumentation would result in increased moments and would thus make the structural limitations of the piles even more restrictive in terms of the amount of load applied to the soil.

b. Studies by Matlock and Ripperger (1957) showed that even a pile and pile cap formed from a solid block of material would not act in a truely "fixed-end" manner. Because no cap, frame, or other conceivable pile-top restraint could act as a perfectly rigid cap, and because the actual restraint could not be accurately measured, the actual end conditions would be relatively indeterminate. A well controlled, end-restraint condition was concluded to be much more desirable than the indeterminate condition with the tops of the piles embedded in a concrete cap.

c. The use of a moment-free connection would be identical with that of a nearby single pile that was loaded laterally and direct comparison of results would be possible.

d. Loading the piles through a frame would allow the insertion of load cells into the frame and thus would

allow measurement of the contribution of each pile to the resistance of the group.

4. The load would be cyclic and two-directional with the period and the variation of the load with time designed to resemble closely that of the loading from a storm on an offshore structure. The load was to be applied using a constant deflection during cycling (and allowing the load to vary) as opposed to using a constant load during cycling (in which deflections would vary). Deflection control had two primary justifications.
   a. It was felt that constant-deflection cycling would have less influence on pile response during subsequent loadings.
   b. The aforementioned load test of the individual pile was conducted in this manner; direct comparison of results requires that similar loading procedures be used.
5. A separate frame for supporting instruments would be used to allow measurement of deflection and slope at the top of the piles. These measurements were needed to derive p-y response properly from the bending-moment data.
6. The sheer volume of data to be acquired and the need for these data to be taken quickly and efficiently dictated that an electronic data-acquisition system be used.

With the above guidelines, the specific design for the test arrangement was worked out; details are described in the paragraphs that follow.

Site Preparation

Shown in Figs. 6.8 and 6.9 are the test-pile group and surrounding pit. This pit was approximately 1.5 ft deep and was kept flooded from October, 1983, to the time of the test in May, 1984. As can be seen in Fig. 6.8, the pit extended for 5 ft (approximately the width of the group) on either side of the group and for distances greater than four times the width of the group in the two directions of loading. The reaction was provided by a drilled shaft that was 6 ft in diameter and with a penetration below grade of 36 ft; the drilled shaft was heavily reinforced. Details of the design of this shaft are presented by Cunningham (1984). An inclinometer tube was placed within the reinforcing cage of the drilled shaft to allow some measurement of the response of the shaft during the test.

Measurements of Bending Moments in the Piles

As mentioned previously, the bending-moment measurements were to be made on an instrumented pipe, grouted into the piles. Due to the extensive instrumentation already present within the piles, the first task was to clean out these piles. In September, 1983, Farmer Foundation Company of Houston, Texas used a truck-mounted drilling rig with a 9-in. diameter coring bit to "drill out" each of the 10-in. ID piles. This drilling was taken to a depth of 26 ft below grade; a snarl of wires and twisted metal was left at that depth. One of the obstructions present in the piles was a 1.5-in.-square tubing that

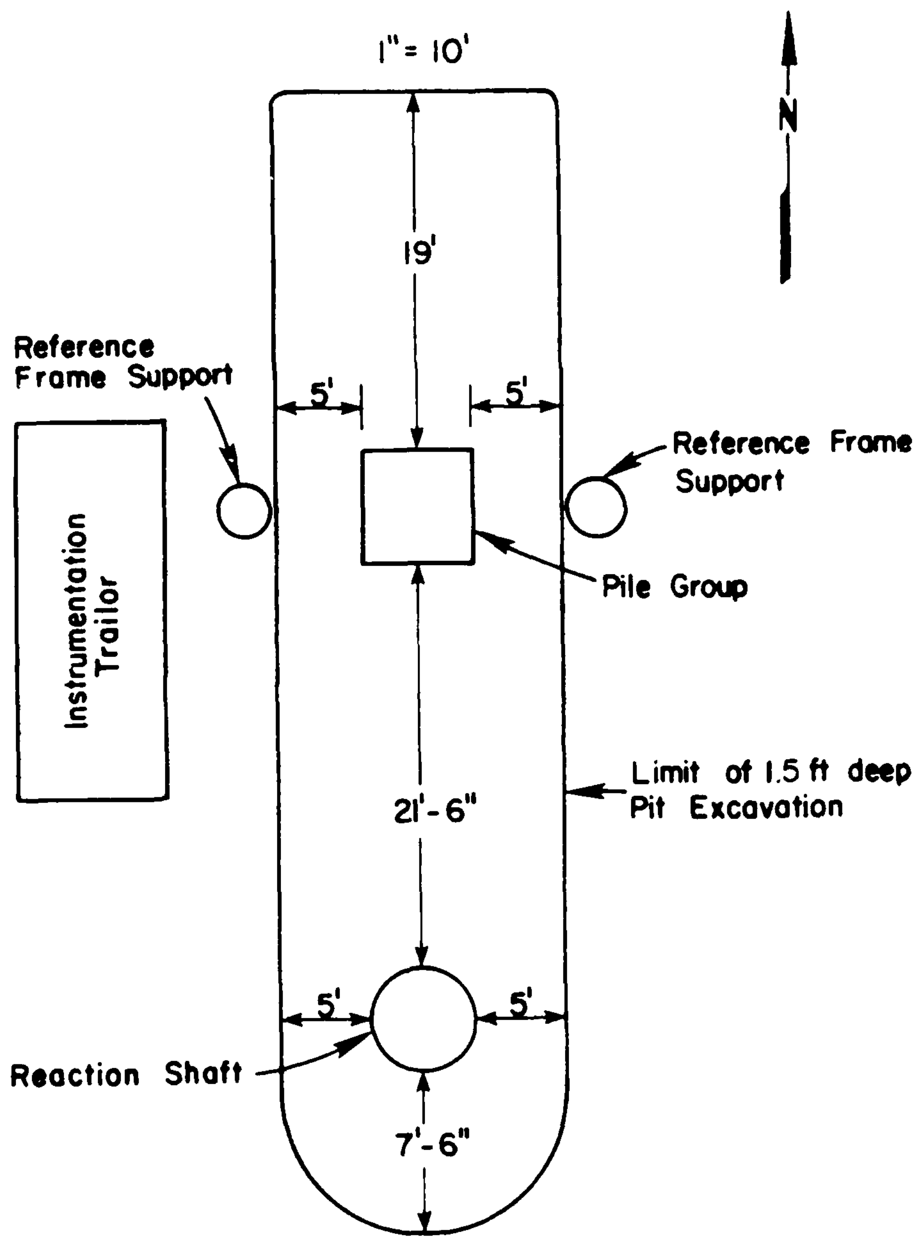


Fig. 6.8 Site Plan

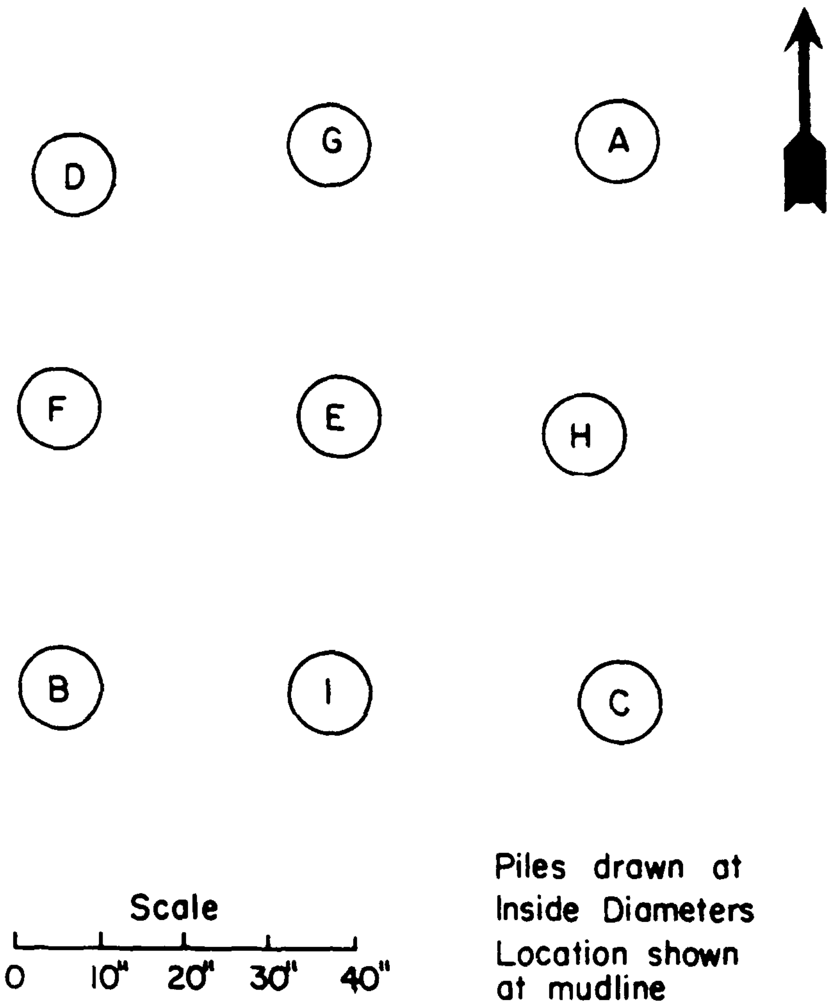


Fig. 6.9 Pile Locations

served as an inclinometer guide during the earlier research. This tubing had been spot welded to the pile at 4 ft intervals. Coring of the first two piles (piles A and H) completely broke this tubing away from the inside of the piles and pushed it to the bottom. Inspection of the remainder of the piles revealed that this tubing was squeezed to the side and not broken loose. This flattening was sufficient to allow insertion of the instrumented pipe without problems and was therefore of concern only for the effect upon the final section modulus of the total section. This effect is relatively small, particularly because most of the tubes were located to the side, near the neutral axis. The instrumented piles were to be calibrated after completion of the test, which would inherently account for any variations in section modulus. No attempts were made to remove anything left after the coring operation with the 9-in. bit.

After coring was completed, a plug was formed atop the snarl in each pile using empty cement bags, large rounded cobbles, and concrete. All of the piles except pile A contained water, which apparently drained into the open tops of the piles from rain ponding on top of the concrete cap. This water was pumped out prior to grouting the pipes into place.

The physical arrangement of pile, pipe, mudline, and gauge locations is shown in Fig. 6.10. The pipes were 6-in., schedule 40, steel pipe, 6.625-in. outside diameter with a 0.310-in. wall thickness. Prior to laying the strain gauges centering tabs were welded onto the pipes to ensure proper alignment within the piles. The "ears" shown in Fig. 6.10 were large U-bolts which served as pick-up

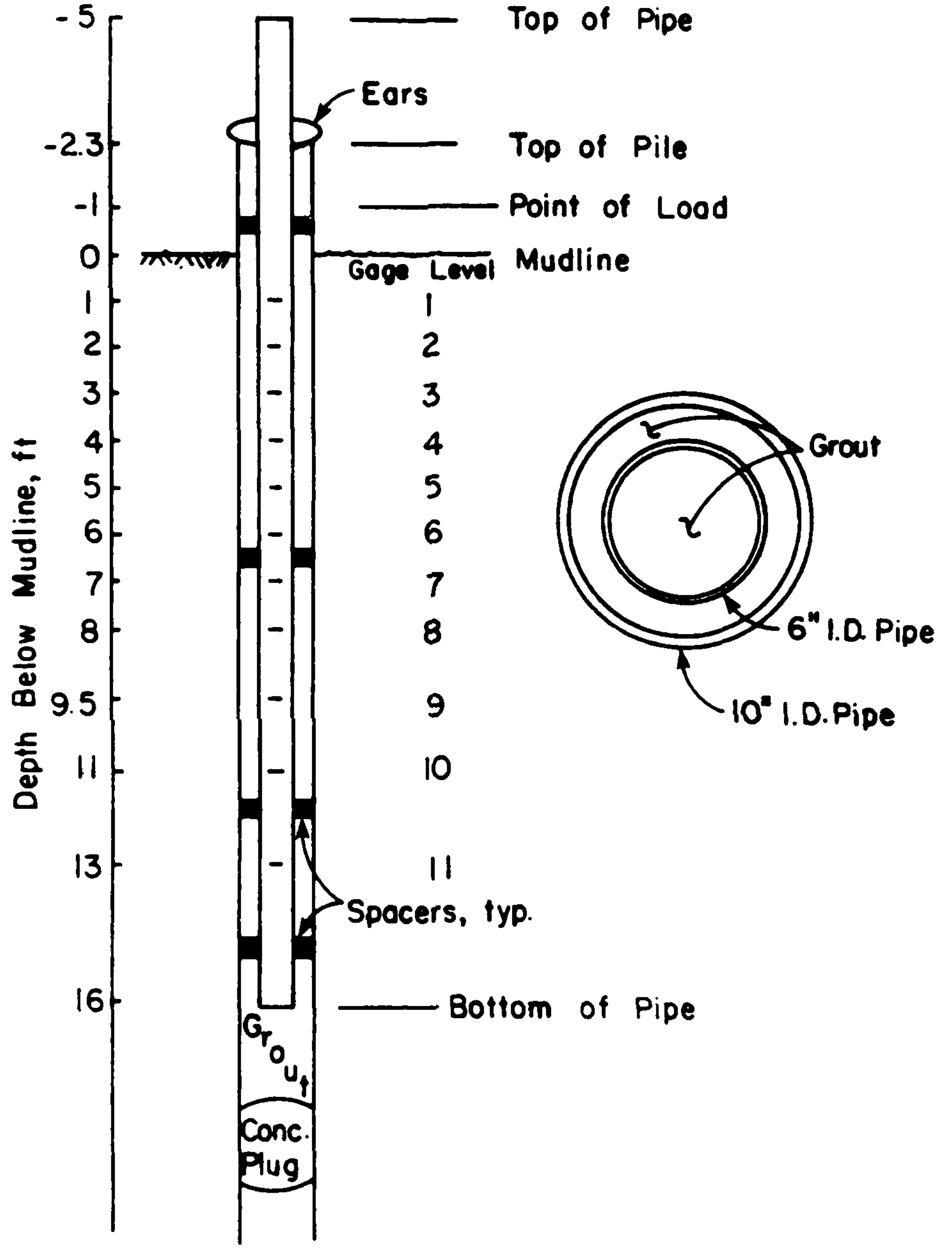
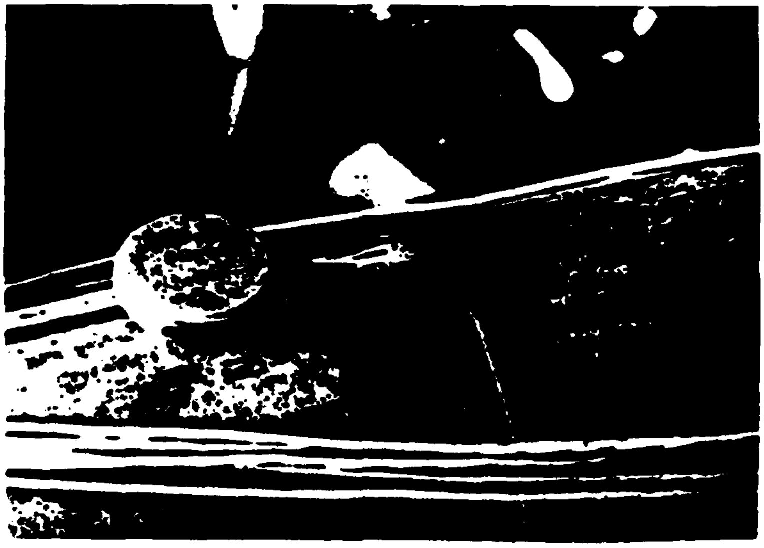


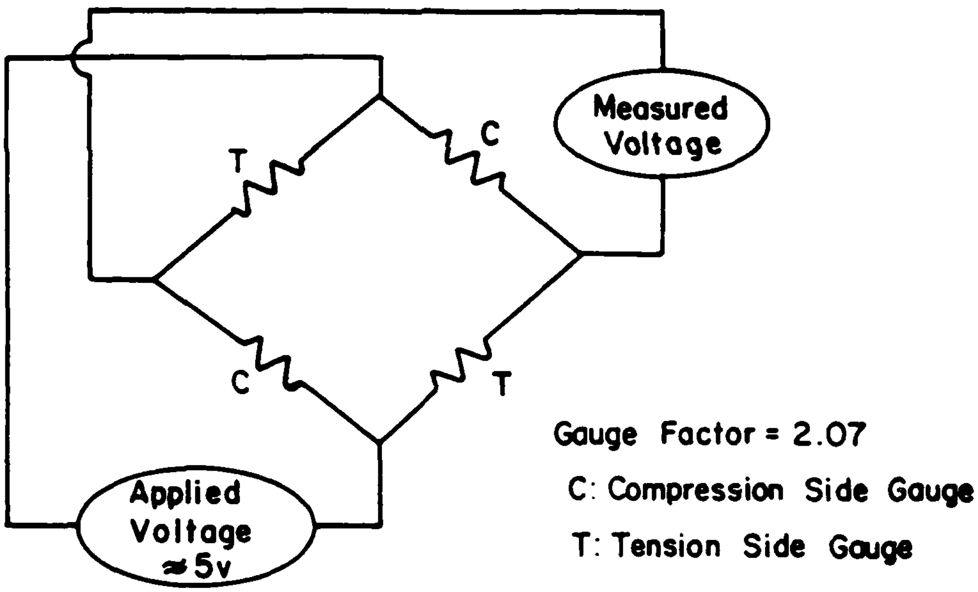
Fig. 6.10 Schematic Drawing of Pile and Instrumented Pipe

points and as stops to support the pipes at the proper elevation. The pipes were marked A through I and inserted in no particular order, and the result is the pile-naming sequence of Fig. 6.9. After placing the pipes, grout was pumped into the center of the pipe and forced to flow upward to the surface through the annular space between the outside of the pipe and the inside of the pile. Figure 6.11 contains two views of the pipe installation. The construction procedure that was adopted resulted in the filling of the inside of the pipes with grout. Although there was no particular desire to fill the pipes with grout, the construction procedure that was used was thought to be the best way to ensure that no voids were entrapped in the annular space. The grout mix consisted of 2 parts of fine sand to 1 part of cement with a water to cement ratio of 0.5. The mix was thus quite fluid, with a slump of about 10 inches. The grouting operations were conducted in March, 1984, and went smoothly. Some grout was spilled onto the ground surface around the piles; this was cleaned up the next day.

The strain gauges were installed on the exterior of the pipes by Norman Peterson of Electronics Measurement Company. The gauges used were Micro-Measurements CEA-06-500UW-120. Each gauge-level consisted of a pair of gauges on each side of the pipe, and all of the gauges were sensitive to strain in the vertical direction. The 4 gauges at each level thus allowed completion of a full-bridge circuit for bending moment; the gauges were connected such as to cancel axial load and temperature effects (see Fig. 6.12). Lead wires for the half-bridge on each side of the pipe extended the length of the pipe to minimize the possibility of snagging a wire during installation of

Fig. 6.11 Placement and Grouting of Instrumented Pipes





$$\text{Strain} = \frac{4}{\text{Gauge Factor}} \times \frac{\text{Meas. Voltage}}{\text{Applied Voltage}}$$

Fig. 6.12 Strain-Gauge Circuit for Measurement of Bending Moments

the pipes and grouting. This method had the additional advantage of allowing completion of the bridge at the control panel for the data-acquisition system; if any gauges were damaged during installation, an external half-bridge could be constructed and thus allow measurement with the remaining half-bridge on the pipe. All gauges survived installation in good shape, so external circuits proved to be not necessary. The lead wires were encased in PVC pipe between an instrument trailer and the test site (a distance of about 25 ft). From the end of the PVC pipe to the top of the grout-filled pile, each bundle of wires was protected by a flexible sheathing made of foam used for pipe installation.

A well-established procedure was followed for attaching the strain gauges to the pipe.

1. The gauge locations were carefully marked on each side of the pipe.
2. All rust and surface irregularities were ground off.
3. A miniature sand blaster and fine emery cloth were used to clean the pipe surface and produce a smooth finish.
4. The surface was cleaned with a neutralizer and thereafter was not touched by hand.
5. The gauge was bonded to the pipe using AE10/15 Epoxy made by Micro-Measurements Corporation. The cement was heat-cured at an established rate.
6. Leads were carefully attached and the entire assembly was coated with Gage Coat 8, a clear epoxy made by Bean Company.

7. Mechanical protection and waterproofing was provided by two layers of Matcoat, a soft epoxy, with a thin brass shim between layers. A final coating of Hysol Epoxy completed the installation.
8. Resistance-to-ground of each gauge was checked; the gauges were removed and the entire procedure was repeated if the resistance to ground was less than 50 megaohms.
9. The ends of the lead wires were tagged for identification. The gauge circuits were completed and checked over a one-day period within one week after grouting and all were found to be working well. The process of checking over a 24-hr period was repeated the day prior to the test. One gauge station appeared to drift slightly; the top level on pile F may have been damaged by some of the welding. Because there was not time to complete an external bridge and check out the new circuit without delaying the test, this gauge level was simply used without modification with the understanding that the data from it are suspect.

Loading Frame and Load Measurement

Shown in Figs. 6.13 and 6.14 are the details of the frame used to transmit load to the piles. The frame was designed to be as rigid as possible, with a maximum deflection between any two piles of less than 0.10 in. at maximum design loads. The hinge-bracket assemblies were purchased from Universal Vise and Tool Company of Parma, Michigan. These units are designed for use as hydraulic-cylinder accessories, but proved ideal for the purposes of this project. The load

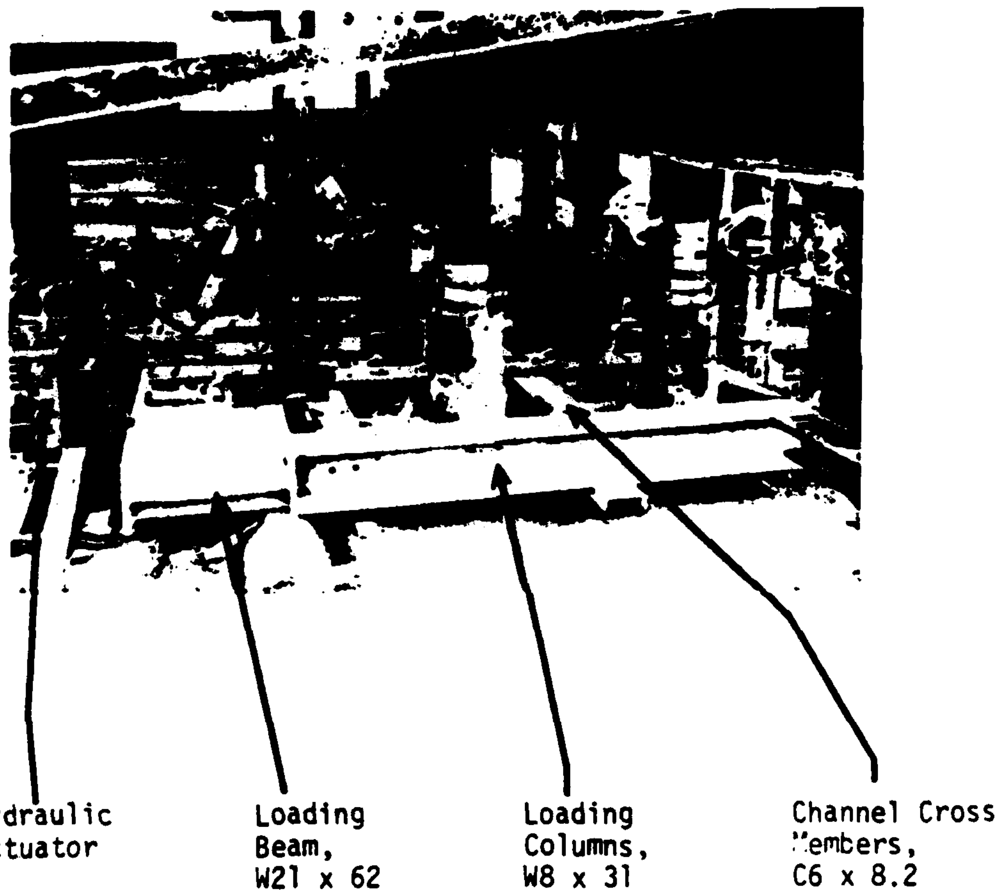
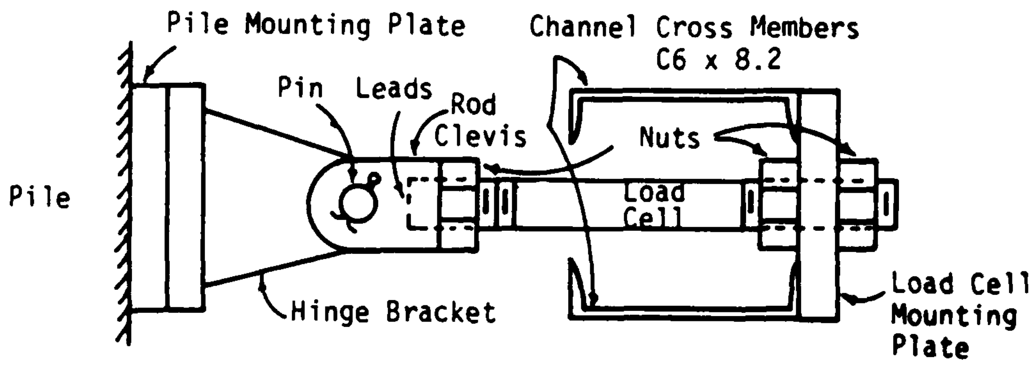


Fig. 6.13 View of the Loading Frame from the East



Load cell assembly

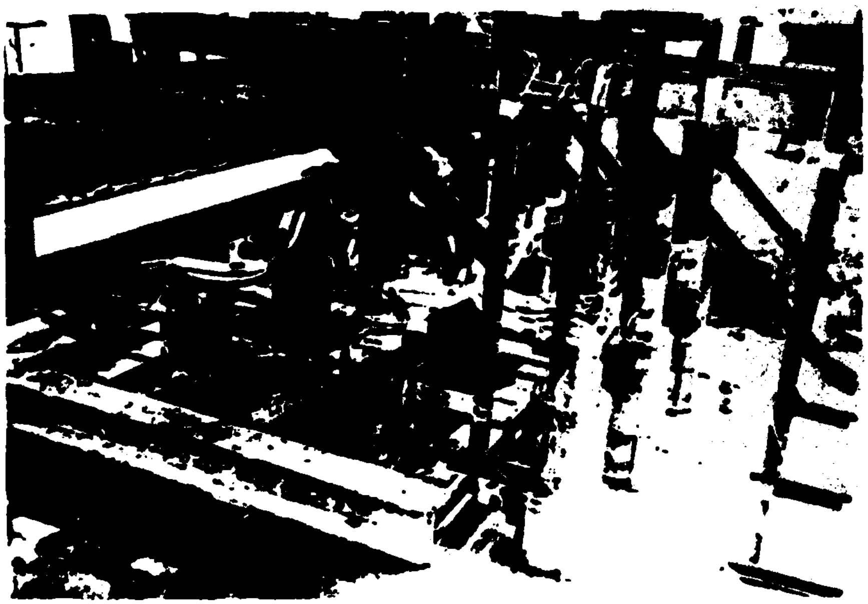


Fig. 6.14 Load-Cell Assembly and View of the Loading Frame from the Northwest

cells were designed to thread into the rod clevis with lead wires extending through the rod clevis and over the hinge bracket. The load cells were constructed in the following manner.

1. A 1.25-in. diameter C1018 cold-rolled-steel rod was cut to the appropriate lengths (11.5 to 13 in. long).
2. Threads were cut on each end to match those of the rod clevis and a hole, 0.25-inch in diameter by 9-inches deep, was bored into the center.
3. A full-bridge, strain-gauge circuit was constructed using a pair of T-rosette gauges that were fastened to a 0.125-in.-diameter steel rod. The strain-gauge circuit was designed to be temperature compensating and to cancel bending stresses, and therefore, to be sensitive only to axial load.
4. The rod was fastened by epoxy cement into the 9-in.-deep hole in the load cell and allowed to cure.
5. The completed load cell was checked for temperature sensitivity and was calibrated.

The following procedure was used to place and connect the loading-frame system.

1. The frame was set into place around the piles and rested on supports that placed the frame at the proper elevation.
2. The plates, containing studs for mounting the hinge brackets, were welded to the piles.
3. The nine short channels to serve as the lower supports for the mounting plates for the load cells for each pile were

fitted and welded into place using the proper load cell and plate assembly for each pile as a placement guide.

4. Load cells and hinge assemblies were bolted into place and bolted to the load-cell-mounting plates.

5. The load-cell-mounting plates were welded to the lower support channels.

6. The upper support channels were fitted into place and welded to the load-cell-mounting plates and to the frame.

7. Most of the supports under the frame were removed.

An additional load cell to be used as a backup was mounted onto the loading ram during testing.

Measurement of Deflection and Slope

Deflection measurements were made by using linear potentiometers of the conductive-plastic type, manufactured by Waters Manufacturing of Wayland, Maine. All of these devices were individually calibrated prior to the load test. These units were mounted on a reference frame as shown in Fig. 6.15. With two vertically-separated potentiometers established above the point of loading on each pile, the slope of the top of each pile could be determined. An extension was added to the top of each pile to provide a vertical separation between potentiometers of approximately four feet. Because no shear force was present in the piles above the point of load, the two deflection-monitoring points could be conveniently located and the deflection at the load point easily calculated. In addition to the eighteen potentiometers used to monitor the nine piles, two potentiom-

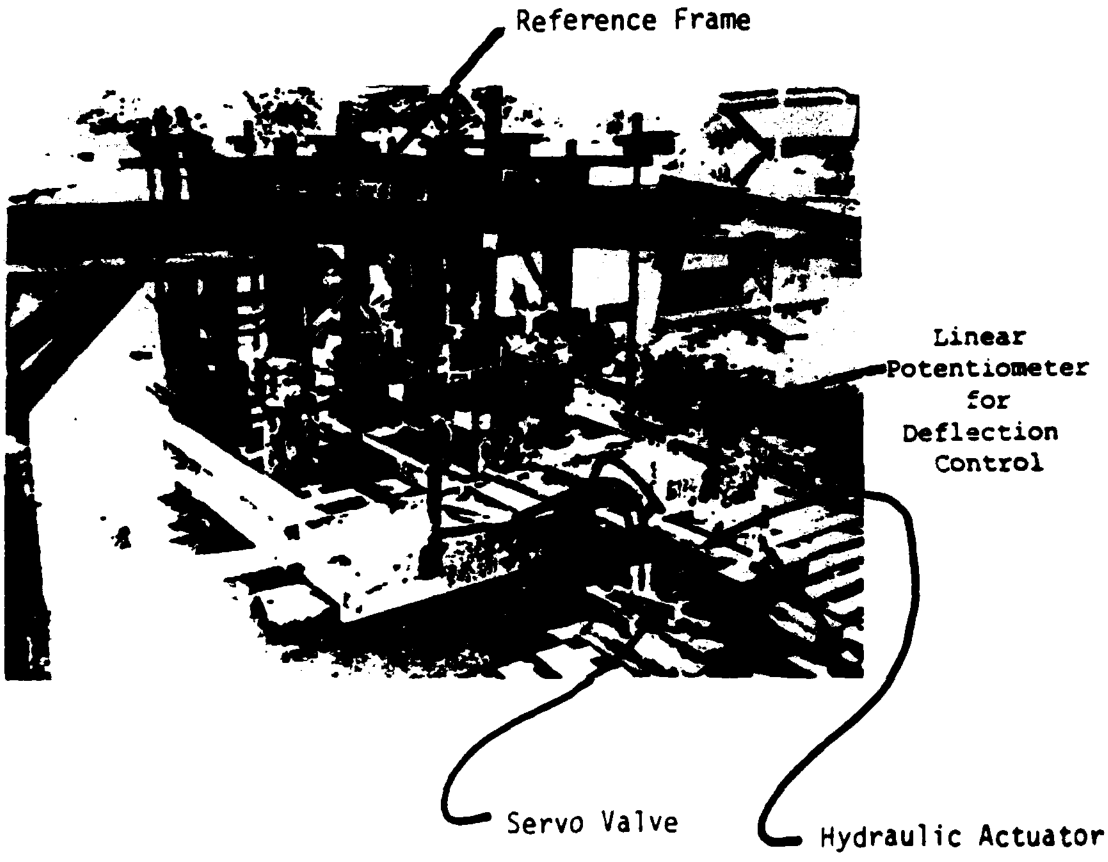


Fig. 6.15 Reference Frame and Loading System

eters were used to monitor the movements of the loading frame and to ensure that relatively plane movements were occurring.

The reference frame shown in Fig. 6.15 was supported on each side of the pit by a 48-in.-diameter steel casing extending to a depth in excess of 50 feet. These casings were used in the earlier axial-load tests to maintain an open hole to the earth anchors founded at a depth of about 100 feet. To minimize temperature distortions of the reference frame, a large tent was erected to provide shade as well as protection from rain. Analyses of possible movements of pile group were made using elastic interaction factors and the procedure outlined by Poulos (1971). These analyses indicated that reference pile movements should be less than .03 in. for the maximum loads applied, which would result in errors of less than 2 to 3%.

Connections between the potentiometers and the pile had to be designed to accommodate rotation as well as vertical displacement as the pile moved. An inexpensive and effective connection was devised using 1/4-in. threaded rod and two joints per potentiometer made of heavy duty air hose and hose clamps. The potentiometers and connections can be seen in Figs. 6.13, 6.14 and 6.15. The air hose was stiff enough to transmit the small axial load in the potentiometer core, but flexible enough to permit rotation. The use of threaded rod and hose clamps provided insurance against slippage.

Data-Acquisition System

Output from the strain-gauge circuits, linear potentiometers, and load cells consisted of analog signals of voltage. These devices were all powered by a single Kepco-regulated direct current, 10 ampere

power supply operated at approximately 5 volts. The signals were monitored and converted to digital signals by 2 Hewlett-Packard 3497A Data Acquisition/Control units controlled by a single Hewlett-Packard 85 microprocessor. This system is shown in Fig. 6.16 in operation during testing. Upon command from the microprocessor, the 3497A units stepped through the 132 channels of data, reading voltage, converting the voltage to a digital signal, and transmitting the signal to the microprocessor. To ensure that the applied voltage from the power supply was not fluctuating during the reading and signal processing, 3 of these channels were dedicated to applied voltage. The microprocessor recorded all data onto a cassette tape, then converted the readings to engineering units for display. The calibration factors that were found by experiment were used for all load cells and potentiometers; estimated calibration factors were used to compute bending moments during the test. The following read-sequence was used:

1. the 10 load cells,
2. the 10 potentiometers,
3. the power supply,
4. the top level of strain gauges in each of the 9 piles,
5. the second through the seventh levels of strain gauges, successively by level,
6. the power supply,
7. the remaining strain gauges, successively by level, and
8. the power supply.

This entire process took about 20 sec, during which the deflection of the pile group was maintained constant. The process of data acquisi-

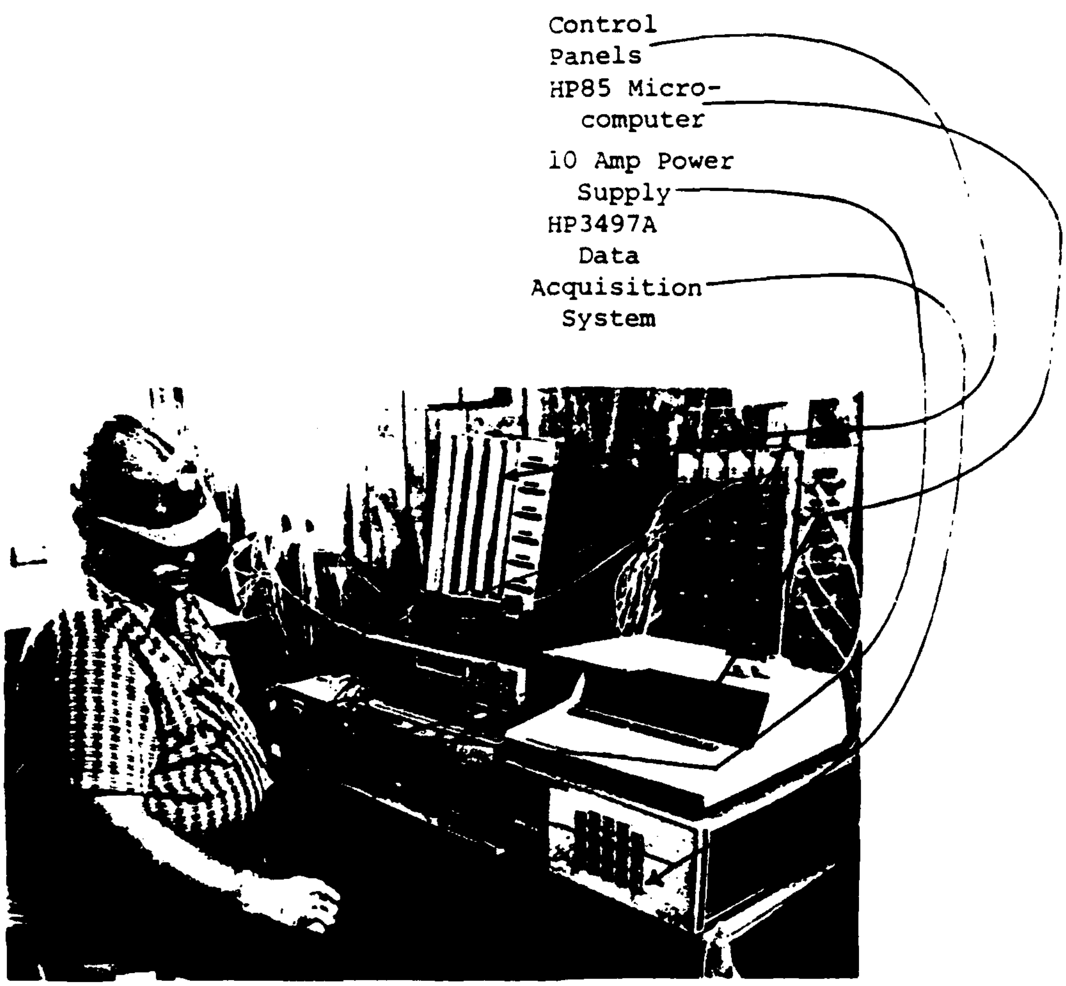


Fig. 6.16 View of Data-Acquisition System

tion could have been performed faster, but at the expense of accuracy. At the speed of other readings and at the level of the applied voltages used in the load test, a sensitivity of 1 x 10^{-6} volts per applied volt was present in the data acquisition.

Load Application and Control

The load during testing was provided by a double-acting hydraulic actuator with a 12-in.-diameter bore, manufactured by the Miller Fluid Power Corporation. The actuator is shown in Fig. 6.15 in place between the loading frame and another frame connected to the reaction shaft. Hydraulic pressure to the actuator was supplied by an MTS model 510.21B hydraulic pump capable of supplying 21 gpm (gallons per minute) at a pressure of 3000 lb/sq inch. Control of the loading operation was maintained by a Pegasus Electro Hydraulic Servo Controller, which operated the MTS Servo Valve shown in Fig. 6.15. An MTS 410 Digital Function Generator was used to provide the desired loading pattern and a linear potentiometer (the same type used on the piles) monitored movement of the loading frame and provided feedback. Feedback was observed on a Dana Model 5403 Digital Voltmeter. The function generator was programmed to output a voltage signal which followed a sinusoidal curve of voltage vs. time (see Fig. 6.17). The period of this sine wave was kept at 30 secs to model that of the loadings from a typical offshore storm (although most major storms have waves with periods that are probably longer); the amplitude was varied during the test. The peak voltages in the positive and negative directions were established to coincide with those desired from the feedback potentiometer. Microswitches were set such that a move-

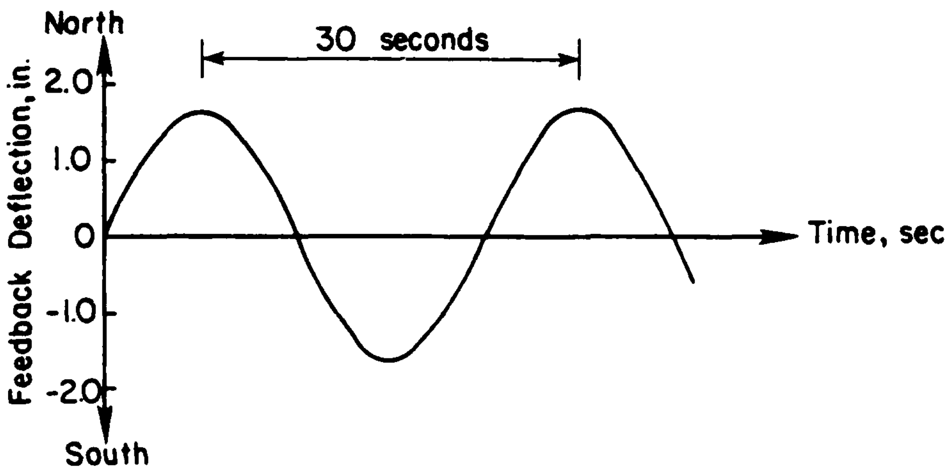


Fig. 6.17 Loading Control, Sine Wave

ment of the loading frame significantly in excess of the expected movement would cause the pump to shut down. When the loading system was started, the Pegasus controller constantly checked the feedback signal vs. the function-generator signal and directed the pressure to the actuator servo valve to maintain a zero difference. The speed with which the actuator could push or pull and thus keep up with the function generator was governed by a 15-gpm flow restriction through the servo valve. With a 30 sec period, 15 gpm was adequate by a factor in excess of 2. The load-control system worked quite well.

Test Procedure

The following procedure was used during testing.

1. A desired load was established and an output voltage from the large load cell for the group to achieve that load was computed. One of the data-acquisition units was set on "local" to display the output from that load cell.
2. Using the Pegasus with manual control (the function generator was thus inactive), load was applied to achieve the desired load.
3. Data were read immediately from all devices and recorded as cycle 1; compression was assumed to be positive for a load applied to the north. At the same time, the output from the feedback potentiometer was noted. The difference in feedback voltage from the zero-load position was input as the amplitude of the sine wave.
4. Tension loading was applied manually, readings again taken, and recorded as cycle 1 tension (south).

5. The function generator was activated and control passed to the Pegasus; cycling at constant peak deflection proceeded.
6. Load was held to take readings on cycles 5, 10, 20, 50, 100, and 200 in both compression and tension directions.
7. After cycle 200, the next load was established and the process repeated.

OBSERVATIONS DURING LOAD TESTING

The test was begun at about 9:00 a.m. Thursday, May, 17, 1984. Readings of the instrumentation were taken just prior to commencing the test to establish zero values. All subsequent data were compared to these zero values. The test was conducted at 5 load levels, largely in accordance with the planned procedure outlined previously. The first load was cycled only 100 times rather than 200, because little degradation occurred at the small load.

At cycle 46 of the second load level a loud pop was heard. The test was stopped and the frame inspected. A weld had broken at the juncture between the load-cell-mounting plate and the lower-channel beam at pile F. No damage was apparent beyond the broken weld, so it was quickly repaired and the test resumed. The load cell on pile F appeared to be unaffected after the weld was repaired. A similar incident occurred on the 13th cycle of load 4 on pile I. Again, the weld was repaired, the test resumed, and no discrepancies in the load cell could be detected.

Load 5 was commenced like all of the others, but shortly after the first readings were taken, a chain reaction of breaking welds occurred on piles A, G, H, D, and E in the same spot that the previous welds had broken. The microswitches shut down the pump and stopped the test. Apparently, the welding of the mounting plate to the lower-channel flange had resulted in fatigue to those welds due to deflection of the channel flange. The construction sequence used and described previously was designed to allow field fitting of the mount of each load cell; however, it was difficult to weld properly the mounting plate along the web of the lower-channel beam. When the 5 welds broke, some of the load cells were noticeably bent.

The welds were repaired and load 5 was then continued by loading in the tension direction only; the broken welds were not stressed at all during tension loading. Cycling was done between full-tension deflection and zero deflection, with the period decreased from 30 to 15 seconds. The test was thus completed (load 5 was the last load) without further incident. Zero readings were taken after completion to allow some attempt at making sense of the readings of the load cells during load 5; however, these readings should remain suspect. These values of the loads on the individual piles are not necessary for interpretation of other data.

Two of the uppermost potentiometers were moved at the start of load 5 to avoid the possibility of their running out of travel during load 5; readings of these potentiometers were merely referenced to a new zero for the remainder of the test. The test was completed at about 3:00 a.m., 18 hrs after testing had begun. After a period of 1

week, several readings were taken under statically-applied loads to examine the possibility of any soil rebound. The load-test system was then dismantled.

During testing, large gaps formed around each of the piles as illustrated by Fig. 6.18. Substantial clouds of fine-grained sediment were observed to be forced out of these gaps with each cycle of load as water was alternately sucked in and pushed out of these gaps during cycling. Because the sediment pumped out of the gaps was gray and the soil surface predominantly brown, the effect was very noticeable. The day after the test was performed a soupy gray muck some 1 to 2 in. thick was present over the surface of the pit all around the pile group. This gray color was more typical of the silty clay material below a depth of 4 ft (stratum IA in Fig. 6.1) than of the surface clay.

Data from the test are presented in Appendix A. Data on bending moments are computed based upon a field calibration. The calibration was performed after completion of the test and is described in the following section.

CALIBRATION AND ACCURACY OF DATA

Field calibration of the strain-gauge circuits used to determine bending moment was performed to better account for variations in pile stiffness. After completion of the load test, a gap had been formed around the piles to a considerable depth; this gap allowed the top 7 gauge stations to be loaded as a cantilever beam. A rough field calibration was computed by comparing the calculated moment at each

Fig. 6.18 Scour During Loading

gauge location (the shear load at the top times the distance below the loading point) to the gauge output. At a later date, the soil was excavated around the piles to a depth of 9.5 ft and a similar calibration performed. This method of calibrating the piles for the acquisition of data on bending moments is somewhat limited by virtue of the fact that the uppermost gauge stations cannot be loaded to as great a stress as occurred during the load test; thus, some extrapolation was required.

The calibration procedure distinguished local variations in pile stiffness (EI), as well as some variation between piles. Sources of local variation in EI were expected due to the presence of the previously installed instrumentation such as stress cells and inclinometer tubing that were used in the vertical-load study. Additionally, the use of cement grout produces some variation in EI during testing because the grout cracks on the tension side of the pile. Calibration of the piles after testing includes the effects of the cracking which occurred during the last and most severe loading; this calibration is therefore incorrect for the early stages of loading, during which the grout is likely to be less severely cracked. The grout used in the test piles of this group theoretically accounts for about 15% of the total stiffness of each pile; one-half or less of this stiffness may be lost due to cracking, thus leading to possible errors in indicated bending moments at the lower loads. Variations in average pile stiffness among piles is likely due to the presence or absence of the inclinometer tubing that was installed during the previous study.

Another factor that must be considered is the accuracy of the gauges themselves. The gauge arrangement in this test did not allow strain measurements to be made at the extreme fibers of the piles, but rather at a location less sensitive to bending moments. This fact was somewhat compensated for by the use of a bridge with 4 active gauges at each station. Calculated variability in the output of each gauge station due to these limitations accounted for a random error of ±10 in.-kips; this value was in fact the upper limit of variability during the 24-hr monitoring period prior to testing.

In consideration of all of the aforementioned factors which can conceivably introduce errors, it would appear that an accuracy of ±10% in measured bending moment values can be claimed.

CHAPTER 7

SUMMARY OF TEST RESULTS

The actual data from the testing are presented in tabular form in Appendix A. Presented in the paragraphs that follow is a summary of the results, as well as a comparison of the behavior of the group with single-pile behavior.

SINGLE-PILE BEHAVIOR

It is instructive to examine results from tests of pile groups in comparison with the behavior of a single pile that is similar to the individual piles in the group. Lateral-load tests with cyclic loading were performed as a part of another study on two single piles of the same dimensions as the group piles and under similar test conditions. One of these piles was thoroughly instrumented for the purpose of deriving p-y curves from the results. The paragraphs that follow summarize the results of the instrumented, single-pile test.

Presented in Fig. 7.1 are the plots of pilehead deflection and rotation vs. lateral load. This pile was loaded in a manner similar to the pile group, with cyclic loading at a constant pilehead deflection. The loading point for the single pile was also similar to the pile group, at about 1 ft above the mudline. An envelope for the results of the first loading in a series is shown. It was felt that because so many load levels were used, some of the values from the

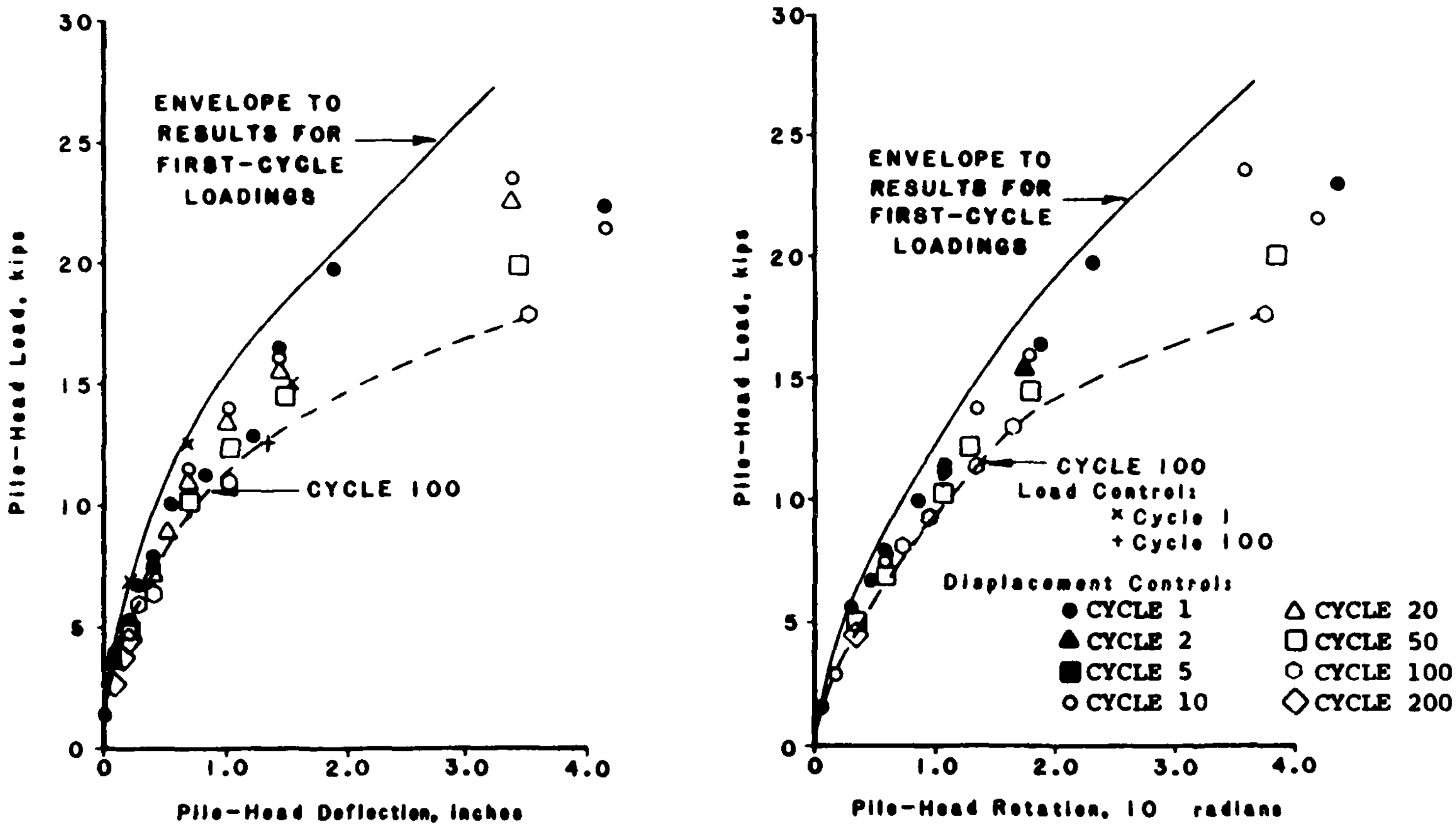


Fig. 7.1 Pile-Head Load vs. Deflection for 10.75-in. Pile During Primary Loading

first load in a series of loads may have been affected by earlier cycling. This upper-bound envelope is thought to be more nearly representative of static-loading conditions. Also, one point is shown at a load in excess of 25 kips. The pile began to deform plastically shortly after the 25-kip measurement was made, resulting in the loss of load at increasing deflection.

Figure 7.2 illustrates the distribution of bending moment with depth for several values of load. The moments have been normalized by dividing by the pilehead load to allow comparisons of static and cyclic behavior. Plots of the measured values produce well-defined bending-moment curves. Curves of soil resistance vs. deflection were derived from the bending-moment data and are presented in Figs. 7.3 and 7.4. The procedures used to derive values of soil resistance and deflection along the pile were similar to those used in analyzing the pile-group data. A description of the methods used to analyze the pile-group data is presented later in this chapter. The p-y curves for cycle 1 appear to show some of the effects of using so many load levels. Some of the values of soil resistance appear to drop off anomalously with increasing deflection; much of the loss of resistance is thought to be attributable to earlier cycling.

The question mentioned previously regarding the effects of earlier cycling on the results from cycle 1 prompted the testing of an uninstrumented but otherwise identical pile using a smaller number of loads. The measurements of pilehead behavior for this pile are superimposed in Fig. 7.1 as "pile 2"; the results agree fairly well with the envelopes plotted in Fig. 7.1.

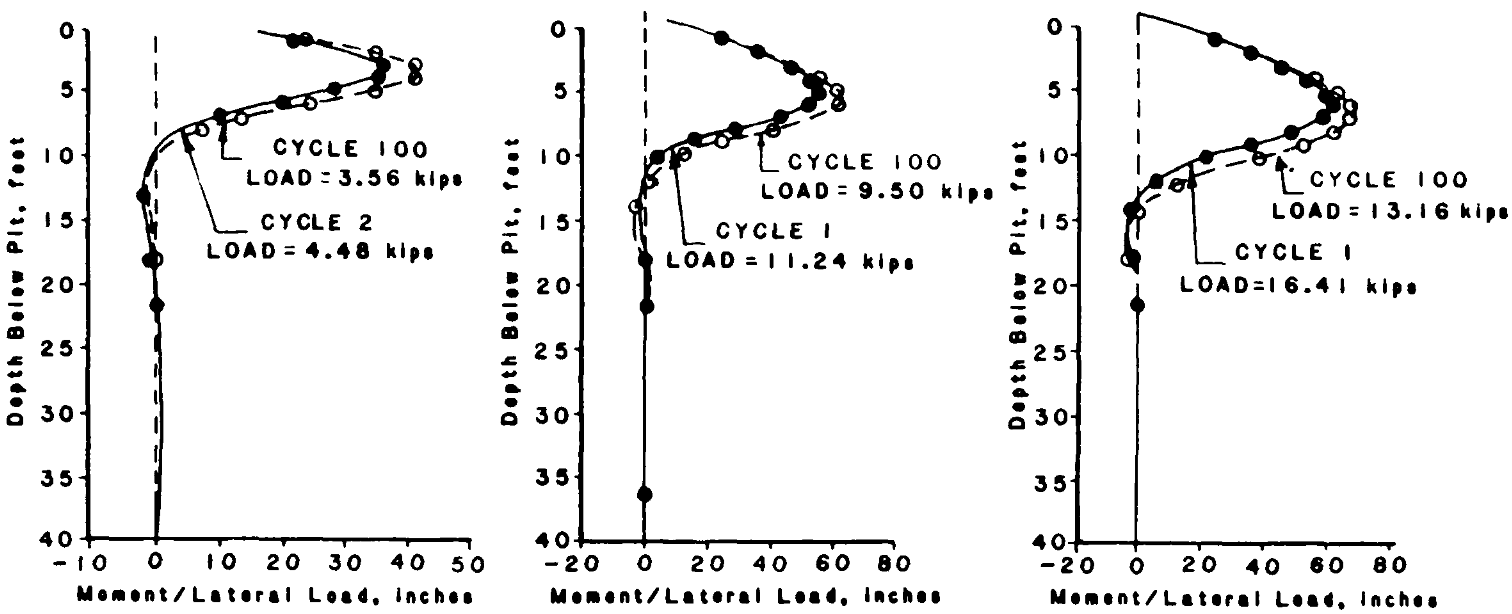


Fig. 7.2 Bending Moment vs. Depth for Single Pile

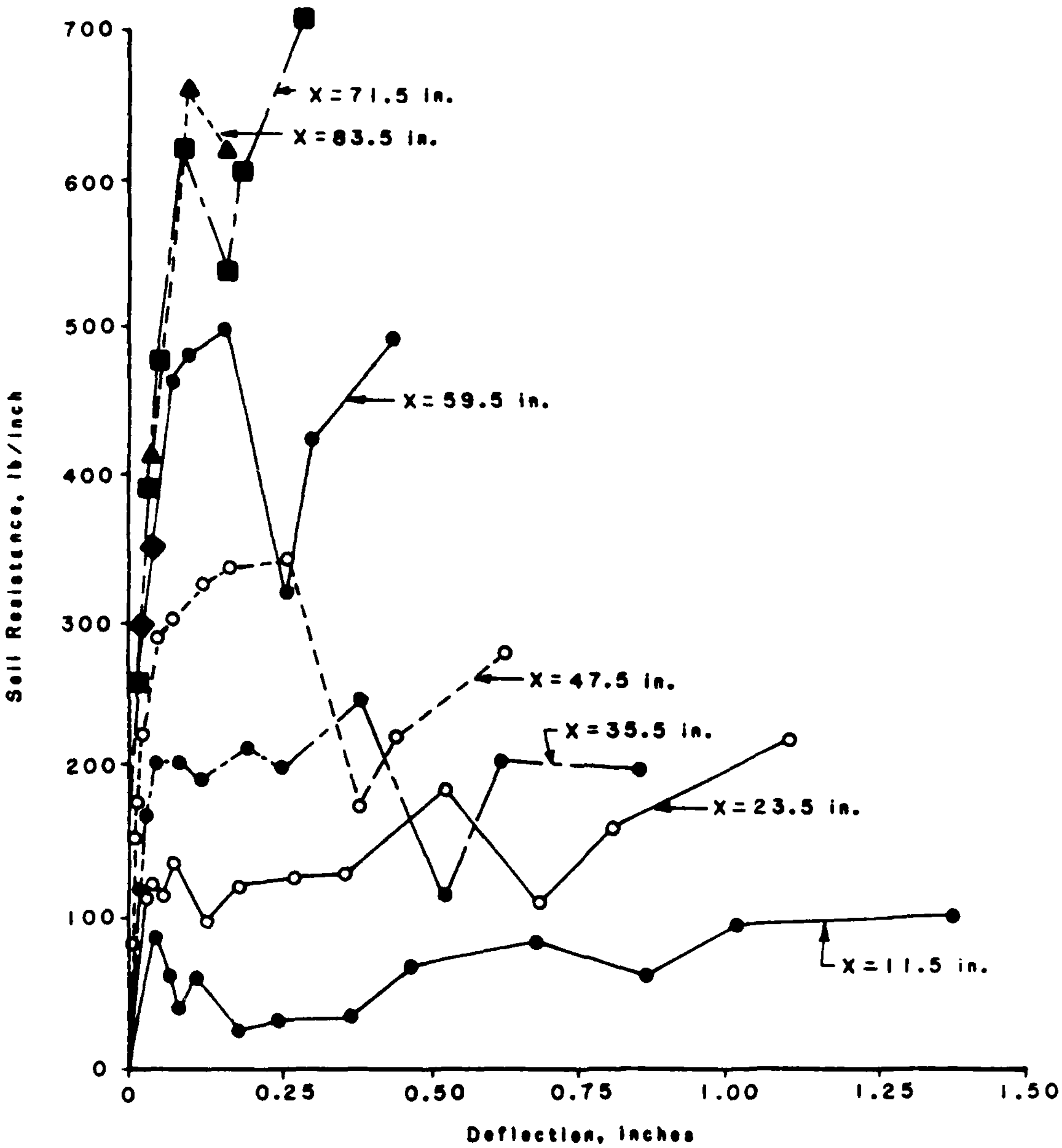


Fig. 7.3 Experimental First-Cycle P-Y Results, 10.75-in. Pile

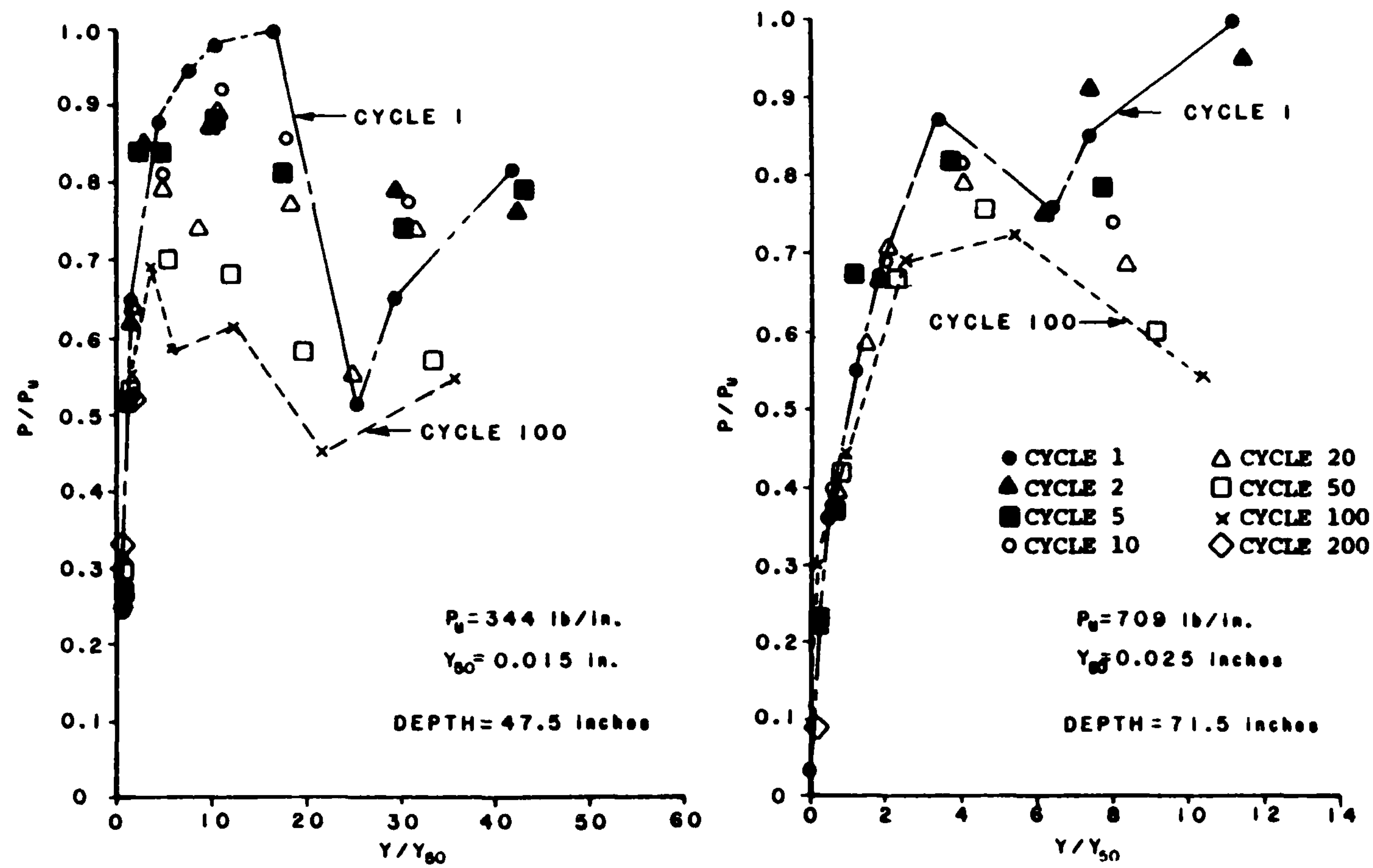


Fig. 7.4 Effect of Repeated Loading on Normalized P-Y Response for Single Pile at Selected Depths

Presented in Fig. 7.5 is a plot of maximum soil resistance (p_u) vs. depth for the single pile, along with a plot of N_p vs. depth. The N_p values were computed using the values of undrained shear strength discussed in Chapter 6. These strengths are based primarily on results of UU tests, as is common practice in the determination of undrained shear strength. The N_p values so computed are seen to deviate considerably from the values suggested by API (1980); the values differ as well from other recommendations shown and referenced in Fig. 7.5. The API equation (after Matlock, 1970) was adjusted to fit these data by modifying the constants of the form.

$$A + \frac{\bar{\sigma}_v'}{S_{u,avg}} + B\,\frac{x}{b} = N_p$$

where:

A and B are constants, and

$\bar{\sigma}_v'$ = effective vertical overburden stress,

$S_{u,avg}$ = average undrained shear strength from the ground surface to the depth, x

x = depth, and

b = pile diameter.

The best fit as shown was achieved using A = 0.8 and B = 0.6.

An explanation for the unusually low N_p values at shallow depths is not immediately apparent; however, the following factors could be of some importance.

1. There was wide scatter in the results from the UU tests at shallow depths.

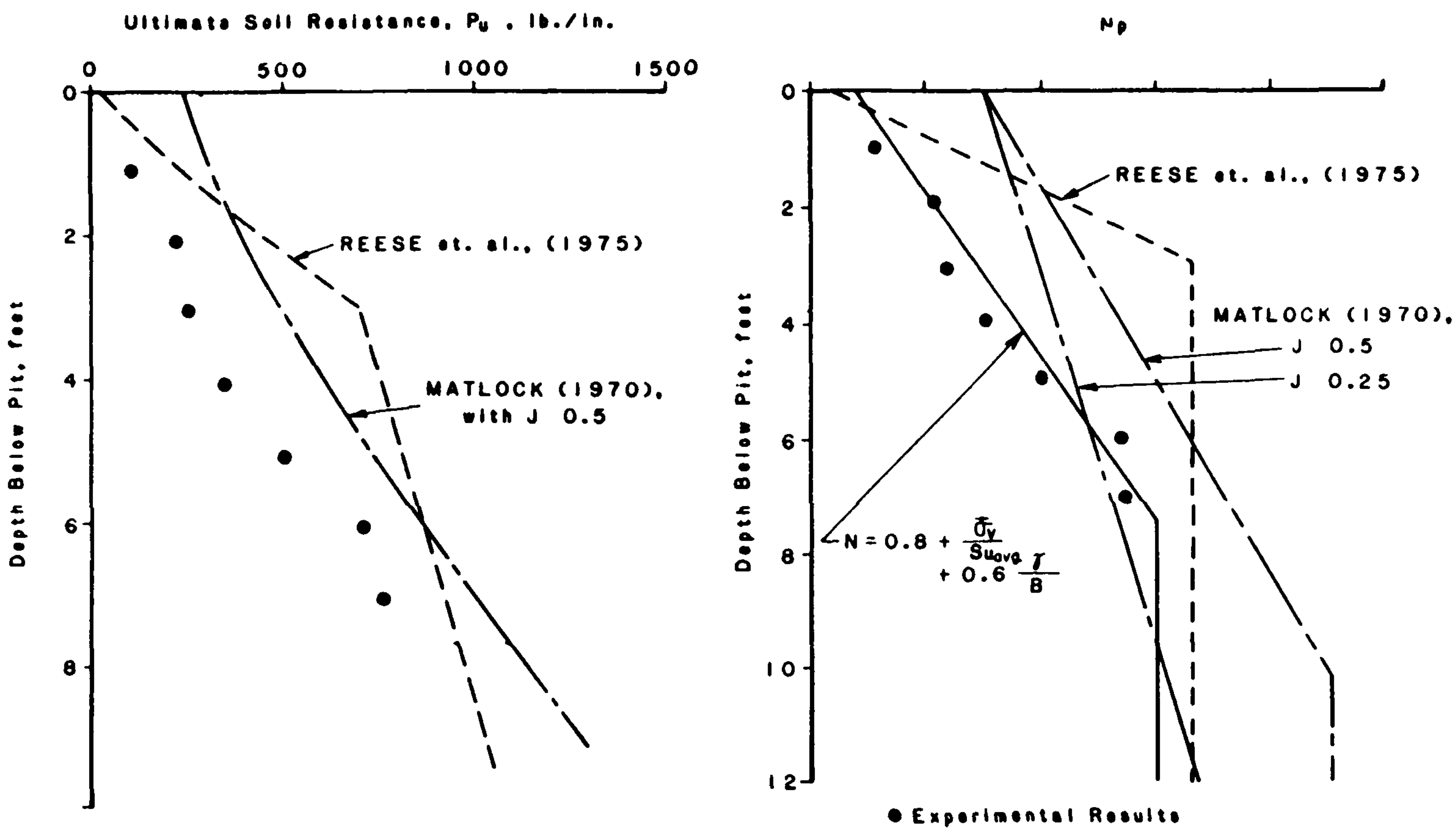


Fig. 7.5 Ultimate Soil Resistance vs. Depth, 10.75-in. Pile

2. Values of undrained shear strength of clay vary widely with test type and soil type. The shear strength of a soil is a function of the effective stress at failure; there is no reason to believe that the effective stress at failure in a UU test in the laboratory is going to be similar to that at failure in the field under the stress imposed by a laterally loaded pile. Existing correlations of p_u and S_u are largely empirical and were derived in different soils. Anisotropic stress and anisotropic shear strengths affect the relationship between shear strength from UU tests and that mobilized during the field test, and these factors undoubtedly vary widely between different soil deposits.

3. The soil at this site had a secondary structure that could affect shear strength as well as drainage in the field. This structure could not be expected to affect laboratory tests to a similar degree. Drainage could be especially important, because laboratory testing is normally performed under undrained conditions.

The above factors serve to illustrate the difficulty in predicting the behavior of a pile group; the state of the art is such that the behavior of a single pile cannot be predicted with a great deal of confidence in a soil for which empirical correlations have not been developed. Consideration of the single-pile measurements and the poor correlations of p_u with S_u from UU tests infer that results from UU tests may not provide the best type of measurement with which to pre-

dict p_u. There appears to be a need for further study of the kind of laboratory or in-situ testing that should be used to predict the behavior of piles under lateral loading, a topic that is well beyond the scope of the research study described herein.

The single piles that were tested did not have the same stiffness as the piles in the group, because the piles in the group included an insert pipe that was grouted in place. A procedure for construction of p-y curves was derived from the results of the test of the single pile that is specific to the soil conditions at the site. This site-specific p-y procedure was used with the computer code COM624 to generate expected single-pile behavior for a pile of the same properties as the average pile in the group (the stiffness of the group piles did vary slightly). The plots of single-pile response under lateral load presented in this chapter, therefore, are not actual measurements from a load test, but are predictions based upon a load test of a very similar pile on the same site.

BEHAVIOR OF PILE GROUP - GENERAL

First Cycle (Static)

Presented in Figs. 7.6 and 7.7 are plots of load vs. deflection and load vs. maximum bending moment for a pile in the group and for the single pile. The curves are presented for the first cycle at each load level, for measurements taken in each of two directions. Because a relatively large increment in load level was used between each successive load level, and because cyclic loading was performed at constant deflection rather than constant load, the first-cycle mea-

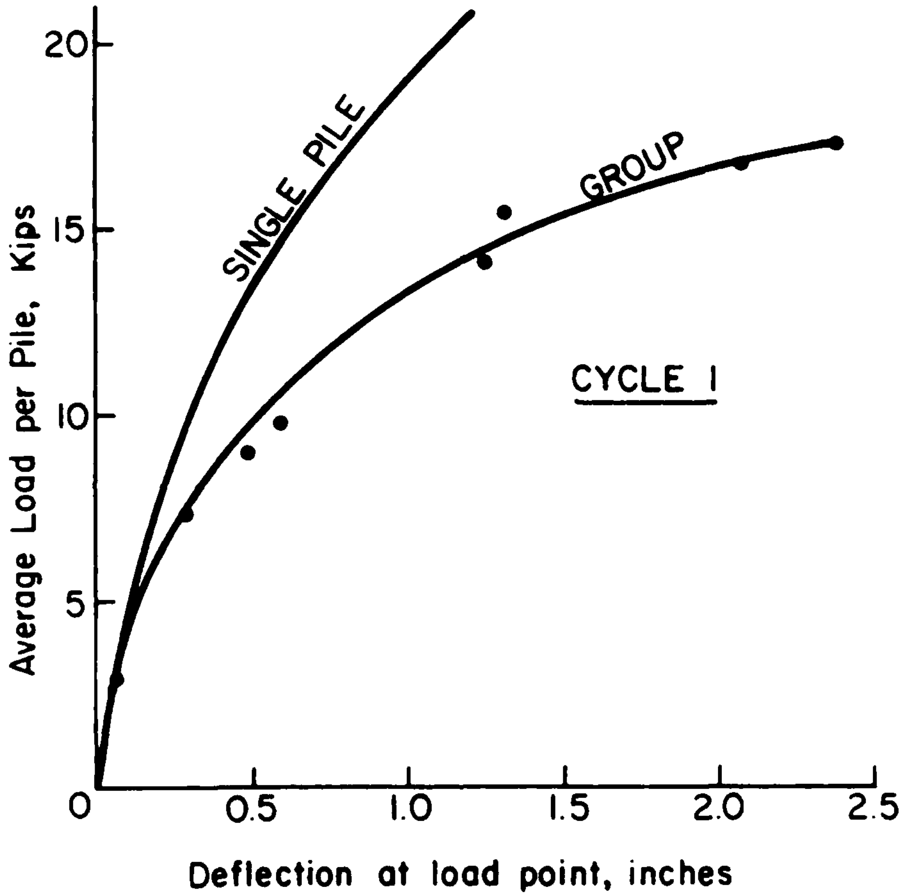


Fig. 7.6 Load vs. Deflection, Cycle 1 (Static)

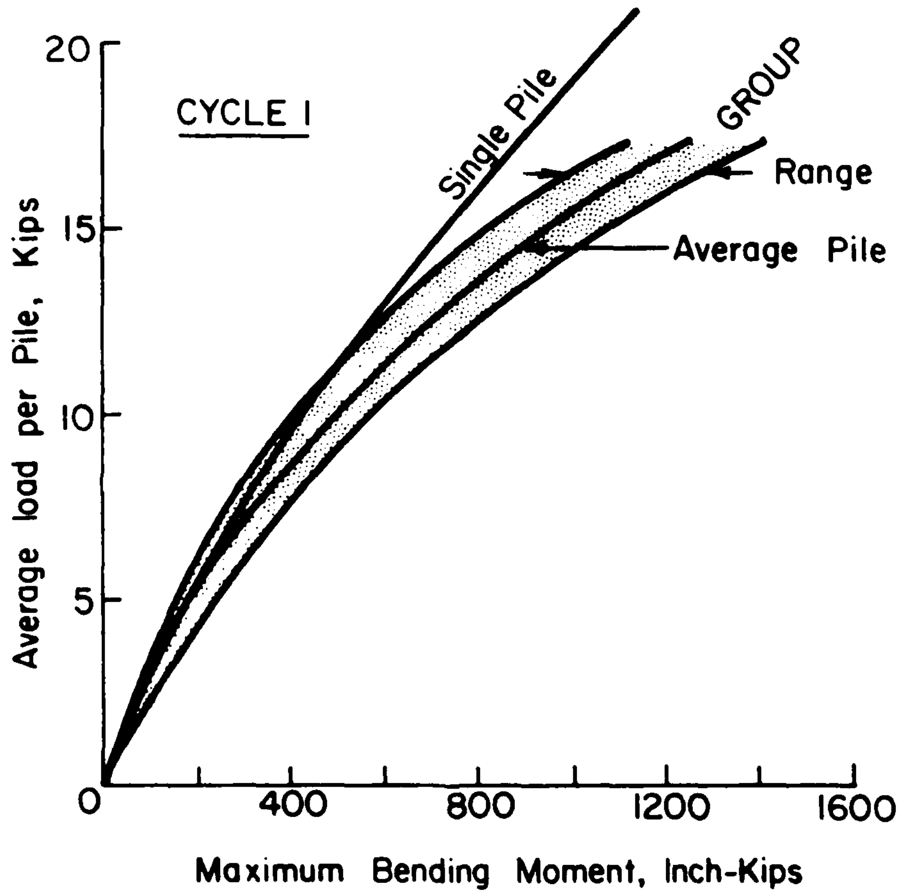


Fig. 7.7 Load vs. Maximum Moment, Cycle 1 (Static)

surements are thought to be representative of behavior under static-load conditions. The plot of load vs. maximum bending moment for the average of the piles in the group is shown as a range to illustrate the variation among the individual piles in the group. The line within that range marked "average pile" is the average in the sense that the bending moments for the individual piles at each gauge station were averaged. Because all piles did not attain the maximum moment at the same depth, it does not therefore represent the average of the maximum moments, but rather the maximum of the average moments. The difference in response between the group and the single pile is as expected, with the pile group acting "softer" in terms of the average load per pile. For loads smaller than about 7 kips per pile, there appears to be very little difference between the response of a pile in the group and that of the single pile. For loads greater than 7 kips, the difference in measured bending moments is less significant than the difference in measured deflection.

Figure 7.8 presents a plot of average load per pile vs. the depth to the maximum moment. The differences in single-pile and group-pile behavior are more evident in this figure, because the maximum moments occur significantly deeper in the case of the piles in the group. These data suggest that the soil near the surface is contributing less to the resistance to load in the case of the piles in the group. This figure has quite significant implications for offshore pile groups, because of the fact that pile-wall thickness is typically tailored to bending moment.

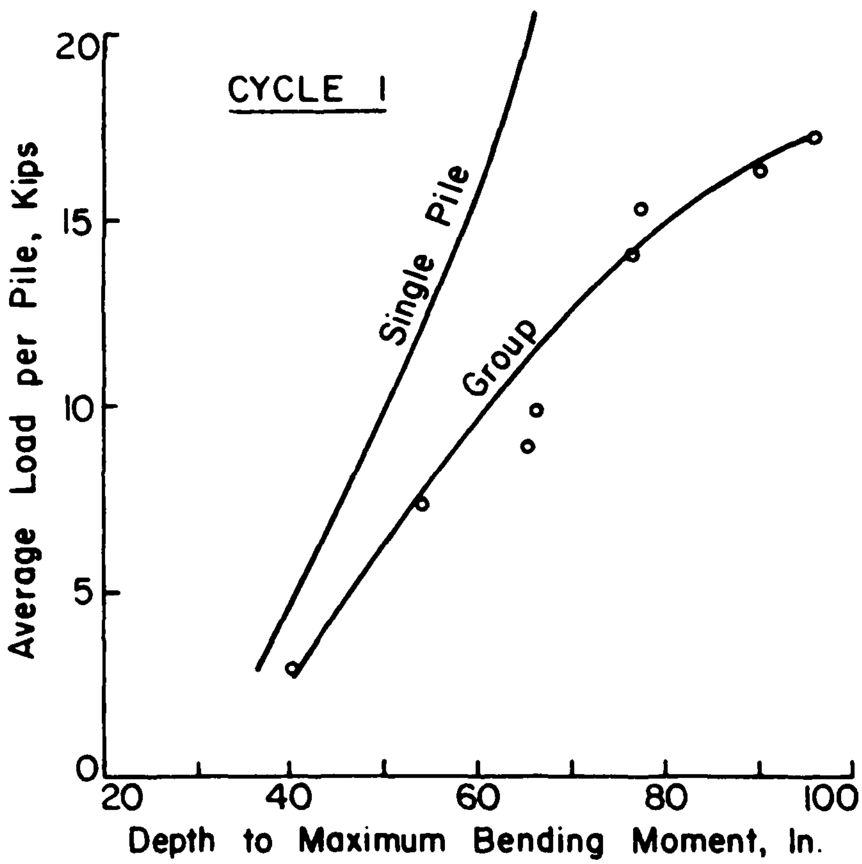


Fig. 7.8 Load vs. Depth to Maximum Moment, Cycle 1 (Static)

Cyclic - General

Figures 7.9 through 7.11 are similar plots of average load per pile vs. deflection, maximum moment, and depth to maximum moment for the 100th cycle of load for the single and group piles. Here the distinction between single-pile behavior and that of the piles in the group is greater and occurs at smaller loads. At first inspection, these data appear to contradict the arguments of several analytical models discussed in Chapter 4 that assumed cyclic degradation to be more of an individual-pile phenomenon, not greatly affected by group action. The differences between the single pile and group piles at cycle 100 are certainly more significant than at cycle 1. However, during the load test it was obvious that the gapping from cycling was confined to an area around each pile of the group. Inspection of the soil around the piles during excavation after the test revealed that these gaps around the piles extended to a depth of around 8 to 8.5 ft, whereas the gap around the single pile extended to only about 6 feet. Apparently, the transfer of load to a greater depth (even under static conditions) results in an extension of non-recoverable strains in the soil to a greater depth and a deepening of the gapping phenomenon for group piles relative to similarly loaded single piles.

Presented in Figs. 7.12 through 7.14 are plots of average load per pile vs. deflection, maximum bending moment, and depth to maximum bending moment for the single pile and the average pile in the group. These plots illustrate the change in behavior during cycling. Both the single pile and the pile group were cycled at constant deflection rather than at constant load. It appears that some loss of load is

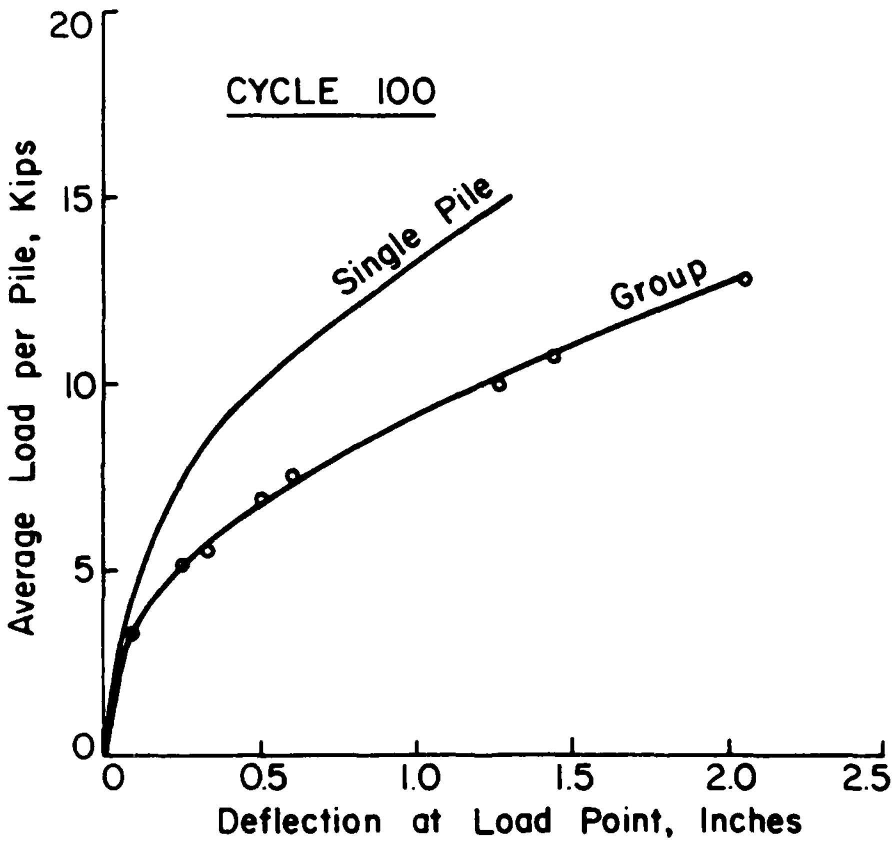


Fig. 7.9 Load vs. Deflection, Cycle 100

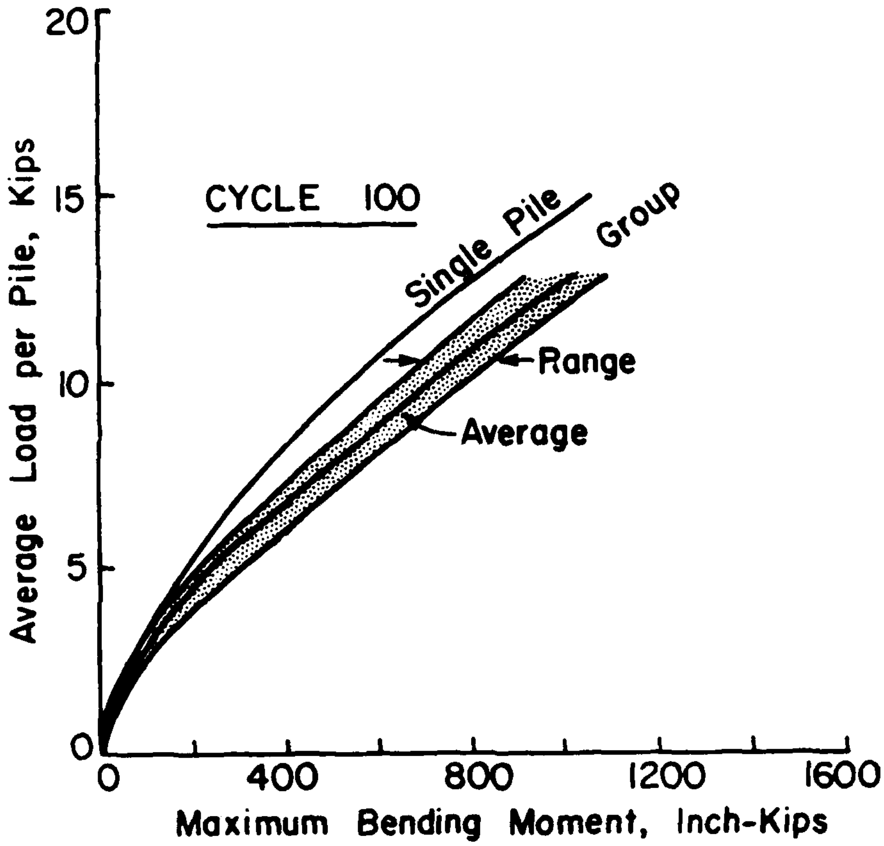


Fig. 7.10 Load vs. Maximum Moment, Cycle 100

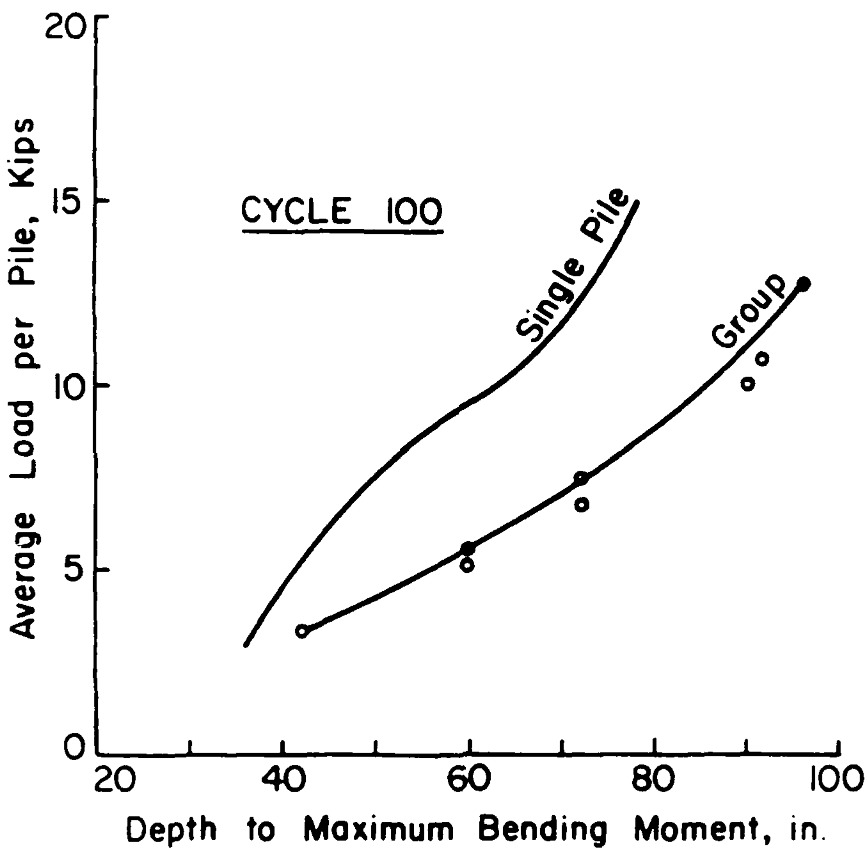


Fig. 7.11 Load vs. Depth to Maximum Moment, Cycle 100

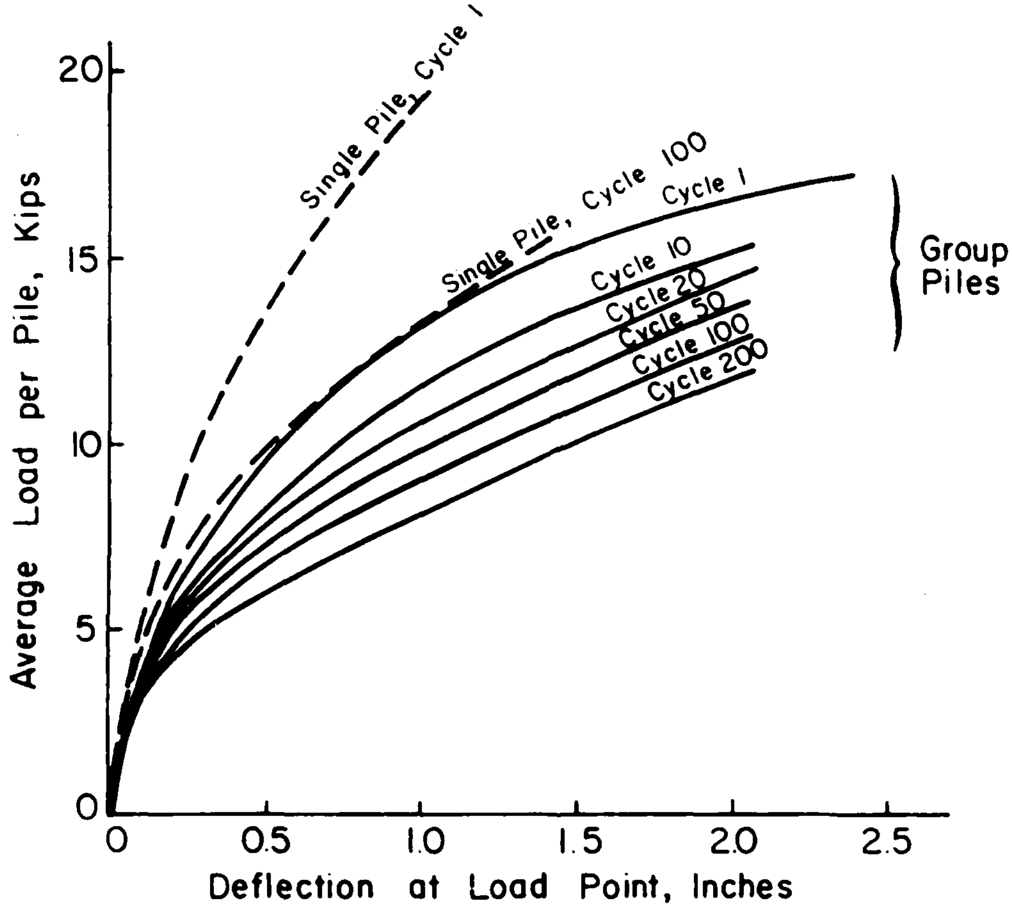


Fig. 7.12 Load vs. Deflection, Cycles 1-200

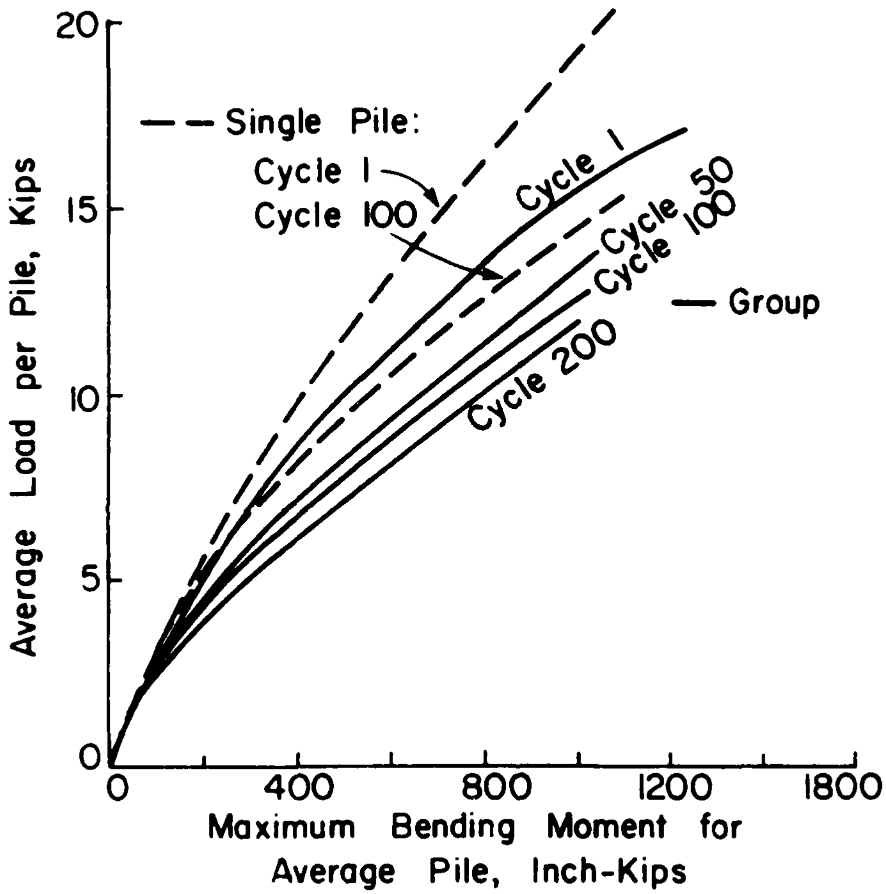


Fig. 7.13 Load vs. Maximum Moment, Cycles 1-200

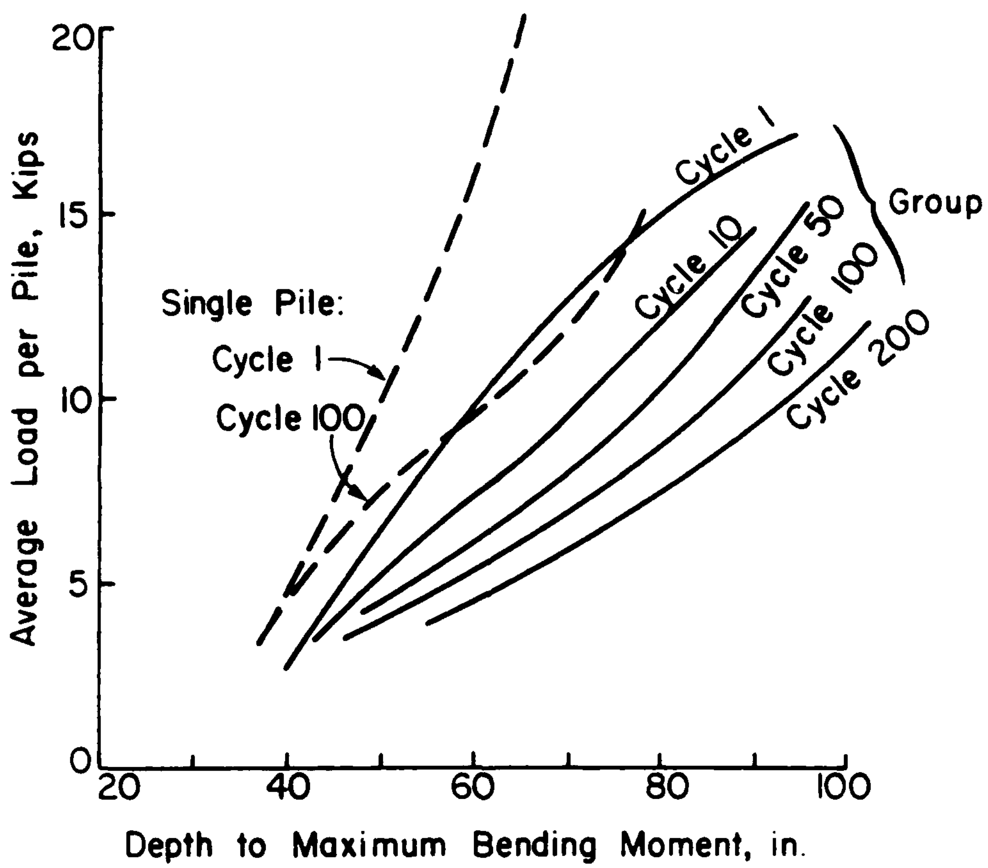


Fig. 7.14 Load vs. Depth to Maximum Moment, Cycles 1-200

occurring even between 100 and 200 cycles, although at a progressively lower rate than the loss at lesser numbers of cycles.

DISTRIBUTION OF LOAD TO PILES IN THE GROUP

The load cells at the point of application of load on each pile provided an indication of the distribution of load to the piles in the group. The distribution of load to the piles appeared to follow a pattern of "front row to back row" rather than one of geometric position as might be expected from elasticity solutions. Presented in Figs. 7.15 and 7.16 are plots of load vs. deflection during cycle 1 (static) for each pile, with the piles grouped by rows. Because the front row in the compression direction is also the back row in the tension direction, data are presented separately for the two directions of loading.

Piles within a given row are observed to behave fairly similarly, with the exception of pile A, the pile in the northeast corner. This pile definitely has a "softer" response to load than piles D and G. Results from calibration indicated this pile to be slightly less stiff structurally, probably due largely to the fact that the coring process used to clean out the previous instrumentation was more effective in this pile. However, the differences in response between pile A and the other front row piles are larger than can be explained by differences in pile stiffness alone. Pile H had a structural stiffness approximately equal to that of pile A, and the differences in response between H and the other middle-row piles is relatively small. It appears likely that variations in soil stiffness and shear strength

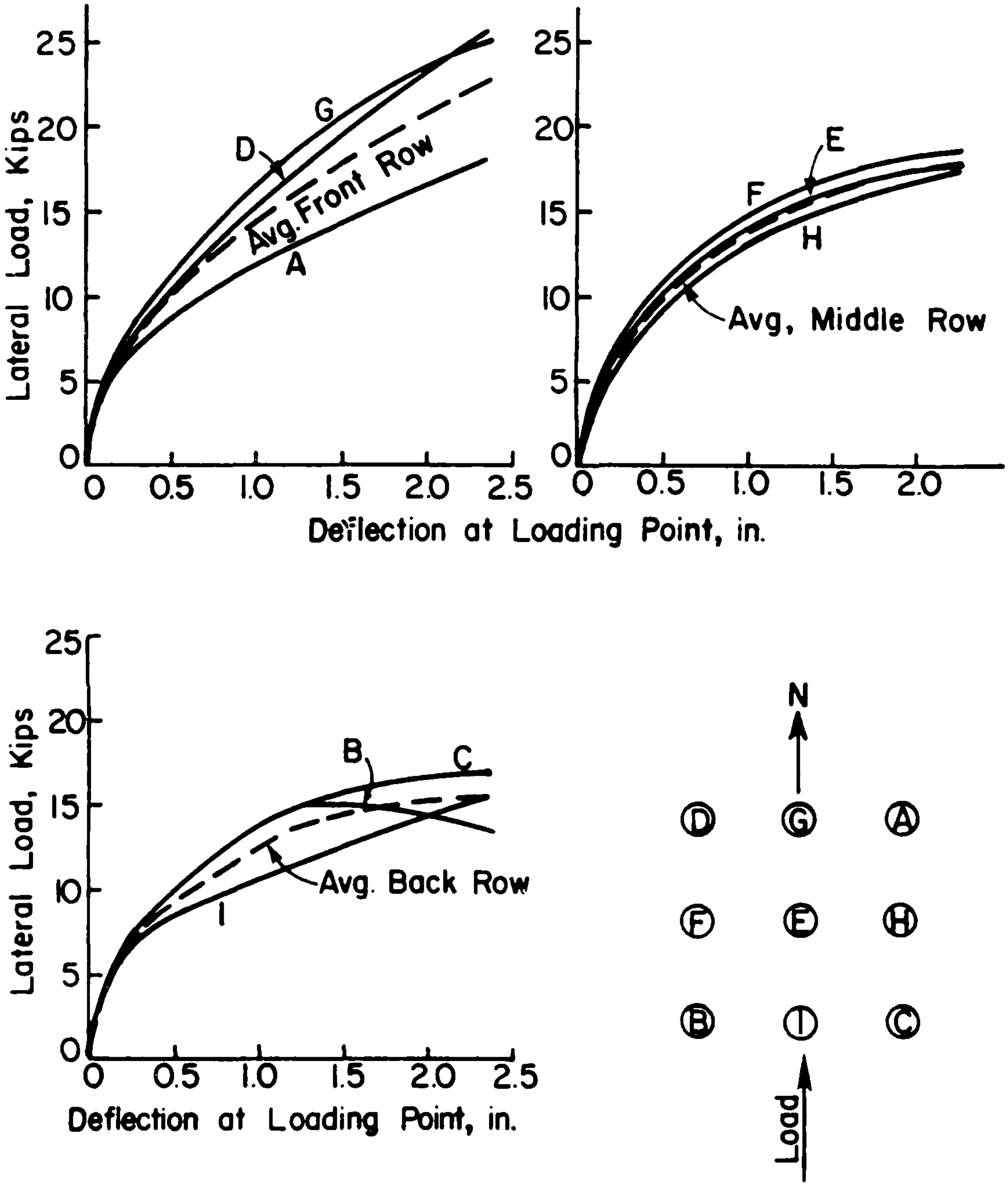


Fig. 7.18 Distribution of Load - Compression, Cycle 1

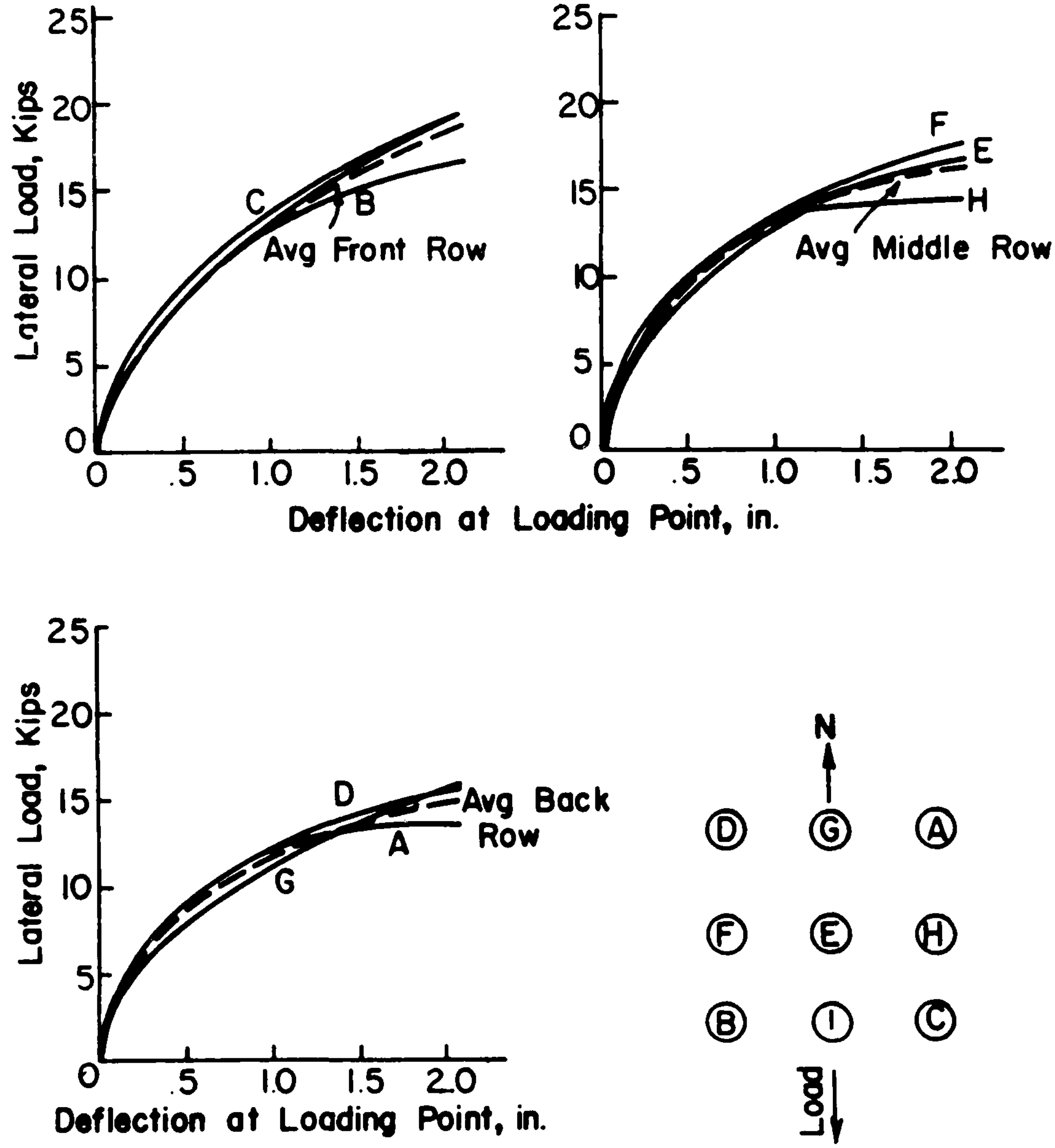


Fig. 7.18 Distribution of Load - Tension, Cycle 1

play some role. Piles A and H were the first piles to fail during the axial-load test in 1979. Discussions with witnesses to the pile-driving operations indicated that pile A was the most difficult to keep aligned during driving, so the possibility exists that soil disturbance from pile driving was greater around pile A.

In order to distinguish patterns of distribution of load in the group, the pile-load data were averaged between compression and tension loads for specific locations and are plotted in Figs. 7.17 and 7.18. As one might expect, the front row carried the most load, followed by the middle row, followed by the back row. A possibly unexpected result is that the center pile generally took the most load on the front row and the least load on the back row. While some of this behavior of the pile in the center of the front row may be due to the anomalous "soft" response of pile A, the trend is present as well in the front row when the loading was in the tension direction. Another item of interest shown in the figure is that the rate of increase of load with deflection in the middle and back rows reduced significantly at higher loads in comparison with the loads on the front row. Between 1.5 and 2 in. of deflection, the front piles were picking up the great majority of the increase in load; apparently the "shadowing" effect was becoming much more prominent.

Similar data are presented for the distribution of load by location at cycle 100. The results are similar to those exhibited for cycle 1; however, except for the pile in the center of the front row, the figure indicates that cycling tends to distribute the load a lit-

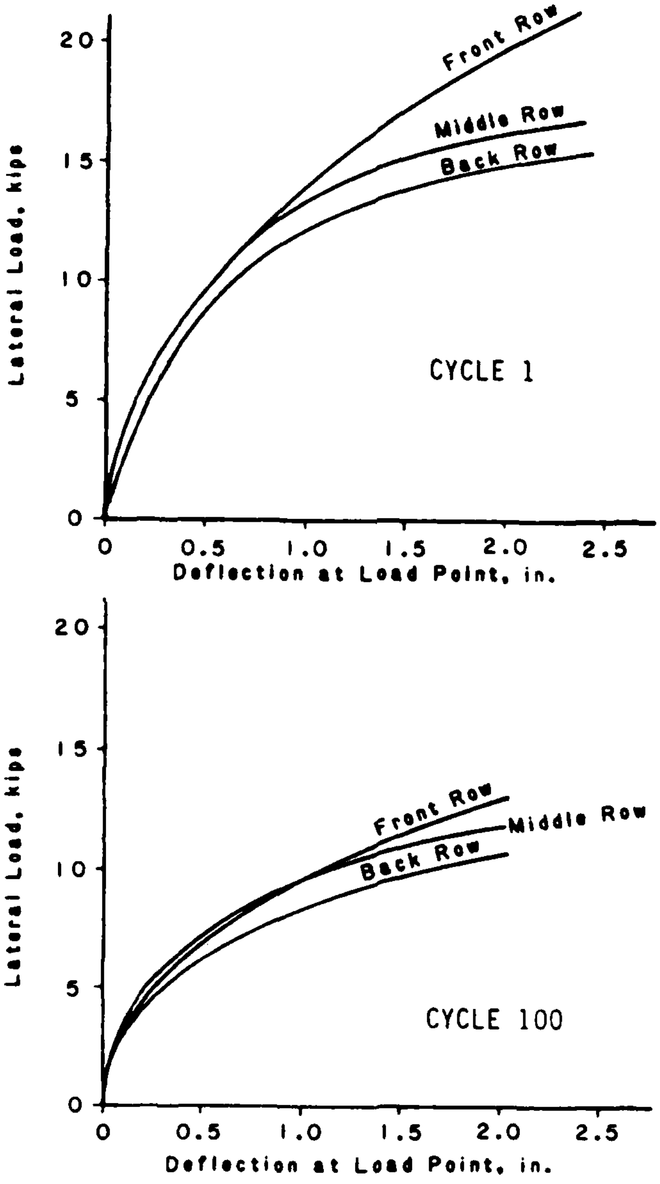


Fig. 7.17 Distribution of Load by Row

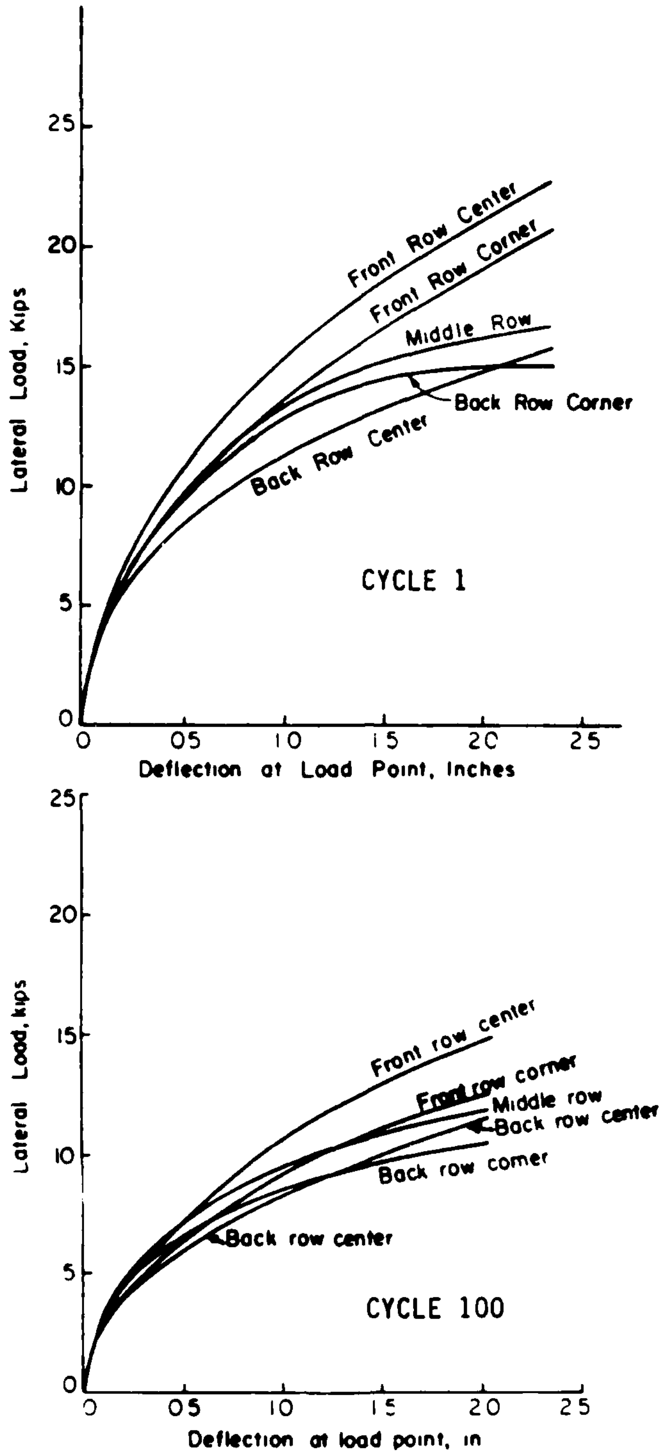


Fig. 7.18 Distribution of Load by Location

tle more evenly at higher loads. The "shadowing" effect at high loads also appears less prominent.

The distribution of bending moment with depth for piles identified by location is presented in Fig. 7.19 for a selected static load for cycle 1 in each direction. The piles in the front row have the largest bending moments in the upper 6 to 7 ft; this result correlates with the fact that the front-row piles had the highest shear at the top. However, the middle- and back-row piles achieve peak bending moments at a greater depth, and the middle-row piles even had a very slightly higher maximum bending moment. These plots are made for the largest loads, where the "shadowing" effect as evidenced by Fig. 7.17 is greatest. The trends evident in Fig. 7.19 suggest that the effects of the "shadowing" may be more prevalent near the ground surface.

It is important to note, when examining Fig. 7.19, that load 5, cycle 1 in tension occurred immediately after load 5, cycle 1 in compression. After loading in compression, it might be expected that the soil stresses and displacements from the middle and back rows under the compression-direction load would tend to cause slightly increased load (proportionally) in the middle and back rows under cycle 1 in tension. The data in Fig. 7.19 reveal a slightly less substantial variation in load between rows for the tension direction (although very slight). Also, the maximum bending moment is at a slightly shallower depth.

Shown in Fig. 7.20 are data similar to that provided in Fig. 7.19, but for a load at cycle 100. The trends for cyclic-load dis-

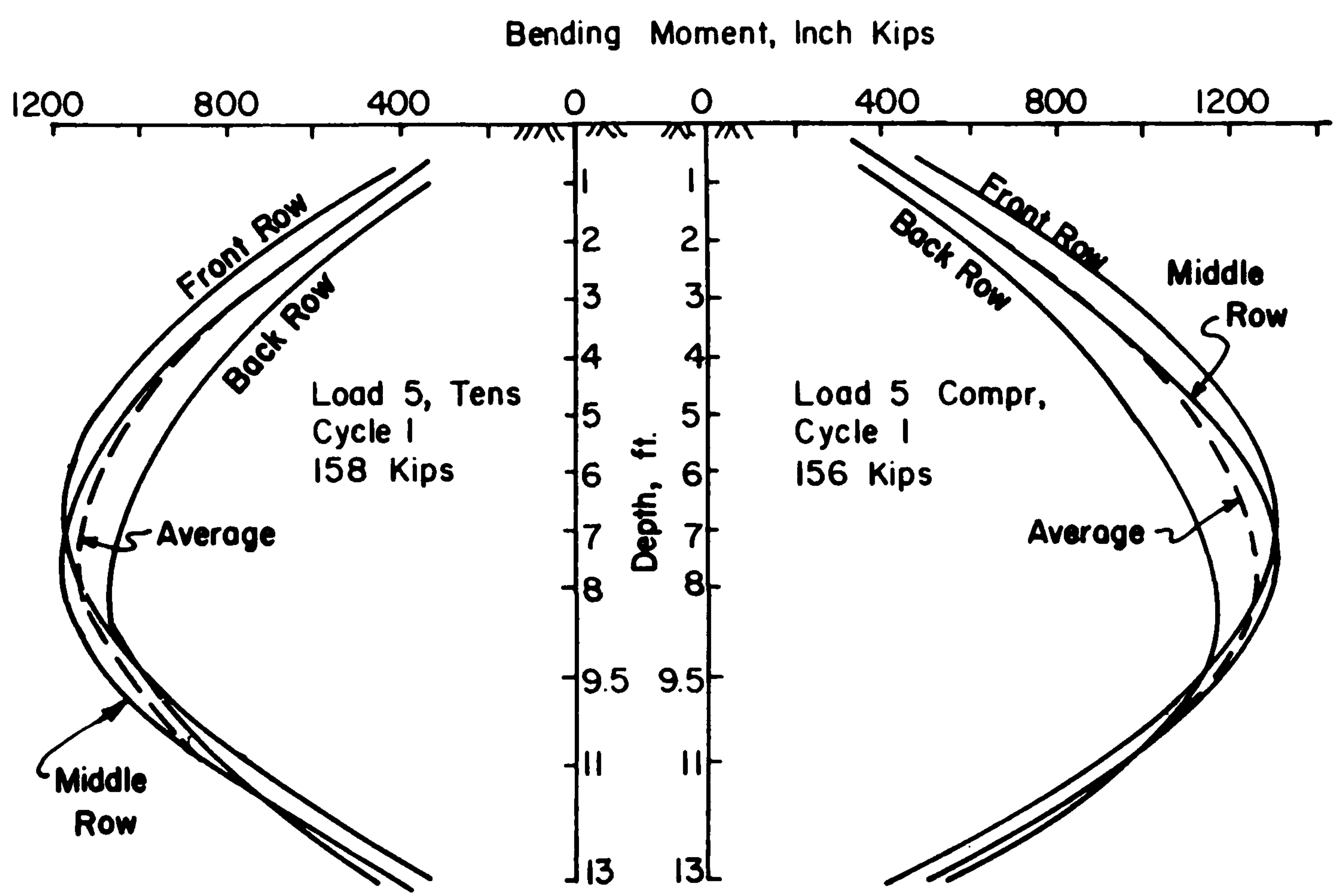


Fig. 7.19 Bending Moment vs. Depth by Row, Cycle 1

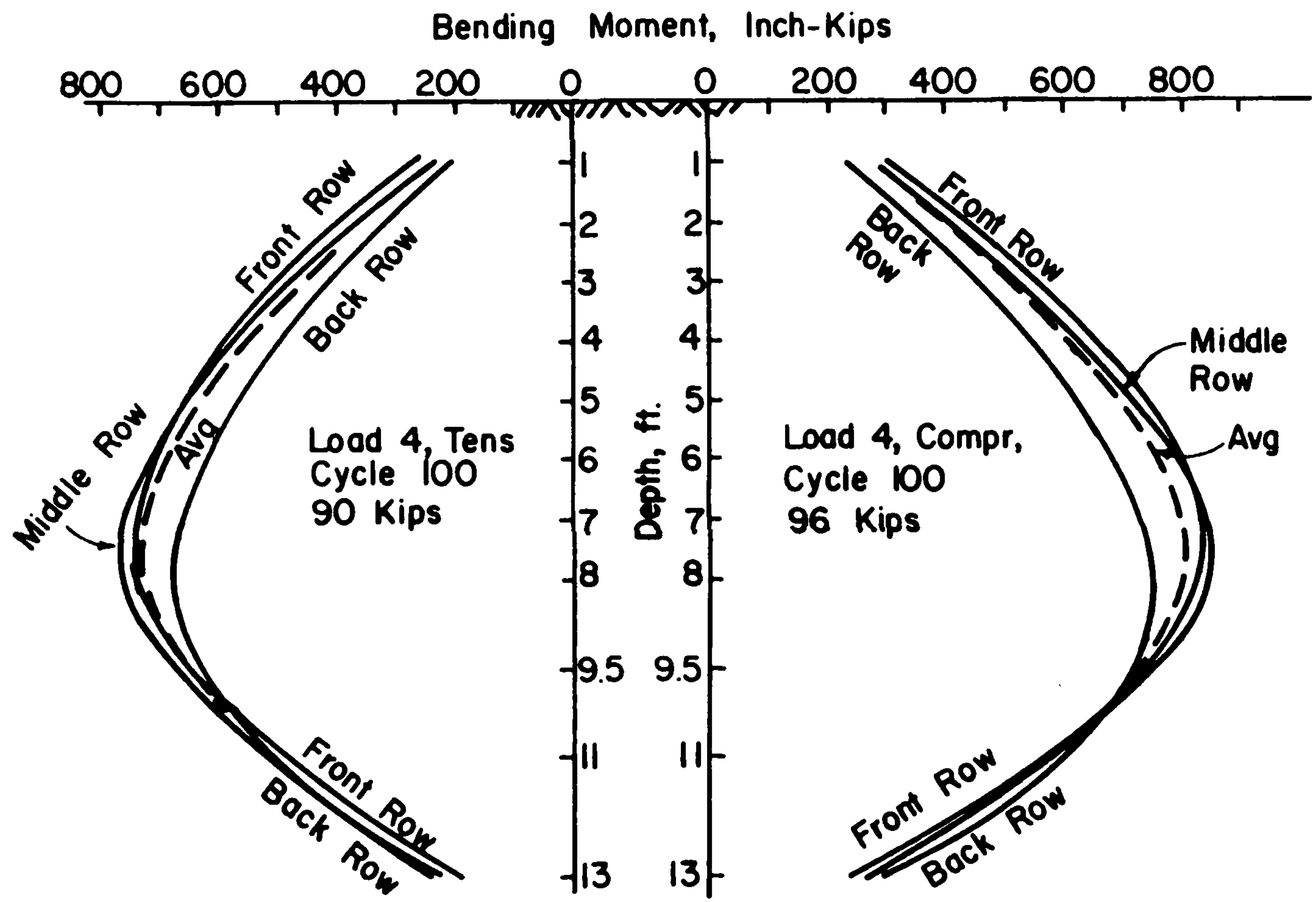


Fig. 7.20 Bending Moment vs. Depth by Row, Cycle 100

tribution are again similar to that for the static case, but less substantial variations between rows are present.

It is of interest to note that the observed load-direction-dependence of the distribution of load to the piles is in contrast to the load distribution pattern predicted using elastic interaction factors. This is a significant result which is discussed in more detail in Chapter 8.

SOIL RESISTANCE

As discussed in Chapter 2, the approach to analyzing the behavior of laterally loaded piles typically involves modelling the soil as a series of nonlinear springs that act independently. Although the model neglects the fact that the soil is a continuum, the errors associated with the approach are assumed to be small. Furthermore, the beam theory incorporated into the model provides a convenient means of analyzing load-test data in which bending moments have been measured.

Method of Analysis of Data

With the soil modelled as a series of springs, the response of a pile to lateral load is described by the familiar fourth order differential equation (presented in Chapter 2).

$$EI \frac{d^4y}{dz^4} + P_x \frac{d^2y}{dx^2} + E_s y - w = 0$$

Where bending-moment data have been obtained, the soil resistance per unit length of pile, p, and the deflection, y, for each nonlinear spring can be computed by the relationships

$$p = \frac{d^2M(z)}{dz^2} \qquad \text{and} \quad y = \frac{1}{EI}\iint M(z)dz$$

The solution for the deflection y requires that two boundary conditions be known, such as slope and deflection at the top of the pile.

Integration and differentiation is facilitated if the bending-moment data are represented by an analytical expression. Although numerous forms for the analytical function are possible, a polynomial expression is simple and straightforward and has been used with some success. When used with the method of least squares, some smoothing of the data can be achieved. After experimentation with polynomials of various degree fitted through a varying number of points at a time, a third-degree polynomial fitted through nine points at a time appeared to yield the desired balance of smoothing and sensitivity.

The polynomial fitting was performed for each pile using data at the point of load application and data from the 11 gauge stations. A fit about a generic gauge station consisted of a least-squares fit with a third-degree polynomial, employing data for the specified gauge station and the 4 stations above and below. Gauge stations 1, 2, and 3, as well as 8, 9, 10, and 11 could not be employed as a center point in this manner and thus the p-y curves derived for these stations were based upon the polynomial fitted about station 4 or 7. The p-y data for stations 1, 2, 9, 10, and 11 must thus be considered less reliable than the others.

As described by Matlock (1958), with the double differentiation of the moment function, the soil reaction, p, is quite sensi-

AD-A193 498 BEHAVIOR OF A LARGE-SCALE PILE GROUP SUBJECTED TO CYCLIC LATERAL LOADING..(U) TEXAS UNIV AT AUSTIN GEOTECHNICAL ENGINEERING CENTER D A BROWN ET AL. 3/5

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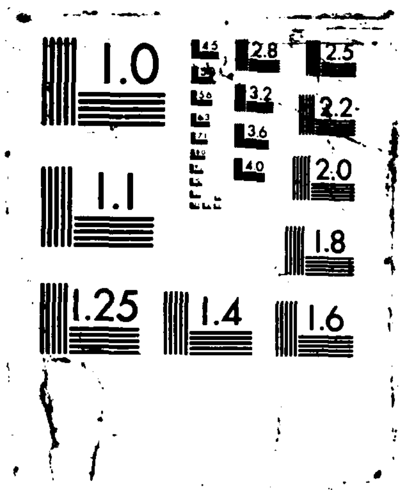

1.0
2.8
2.5
3.2
2.2
3.6
4.0
2.0
1.1
1.8
1.25
1.4
1.6

tive to the accuracy of the bending-moment measurements. Matlock provided an example in which a one percent error in the bending moment produced a seven percent error in the soil reaction. Due to the method of instrumentation and calibration used for these piles, the level of accuracy in the bending moments is likely not as good as that achieved in Matlock's early single-pile tests (Matlock and Ripperger, 1956). In his studies Matlock used a third-degree polynomial fitted through 5 points to achieve a greater degree of sensitivity to variation in soil properties than was possible in this study; the use of a third-degree polynomial fitted through 9 points is not as sensitive to soil properties, but is also less sensitive to errors in bending moment due to the greater degree of smoothing. Double differentiation of a third-degree polynomial results in the assumption of a linear variation of soil reaction over the range of fit.

Results from p-y Curves - General

Presented in Figs. 7.21 through 7.28 are the p-y curves derived from the measurements for gauge stations 1 through 8. Results are presented for cycles 1 and 100 for the average of all piles and for the average of the piles in each row. Piles A, D, and G provided front-row data for the compression-load cycles, while piles B, C, and I provided front-row data for the tension-load cycles. Data from 6 piles were thus averaged to produce each point on the plots shown by row. Data from 18 pile measurements were used to produce the points on the plot for the overall average. An exception is noted for the case of load 5, cycle 100 (the last point on the cycle-100 curves); after the cycle 1 measurements were made in both compression and ten-

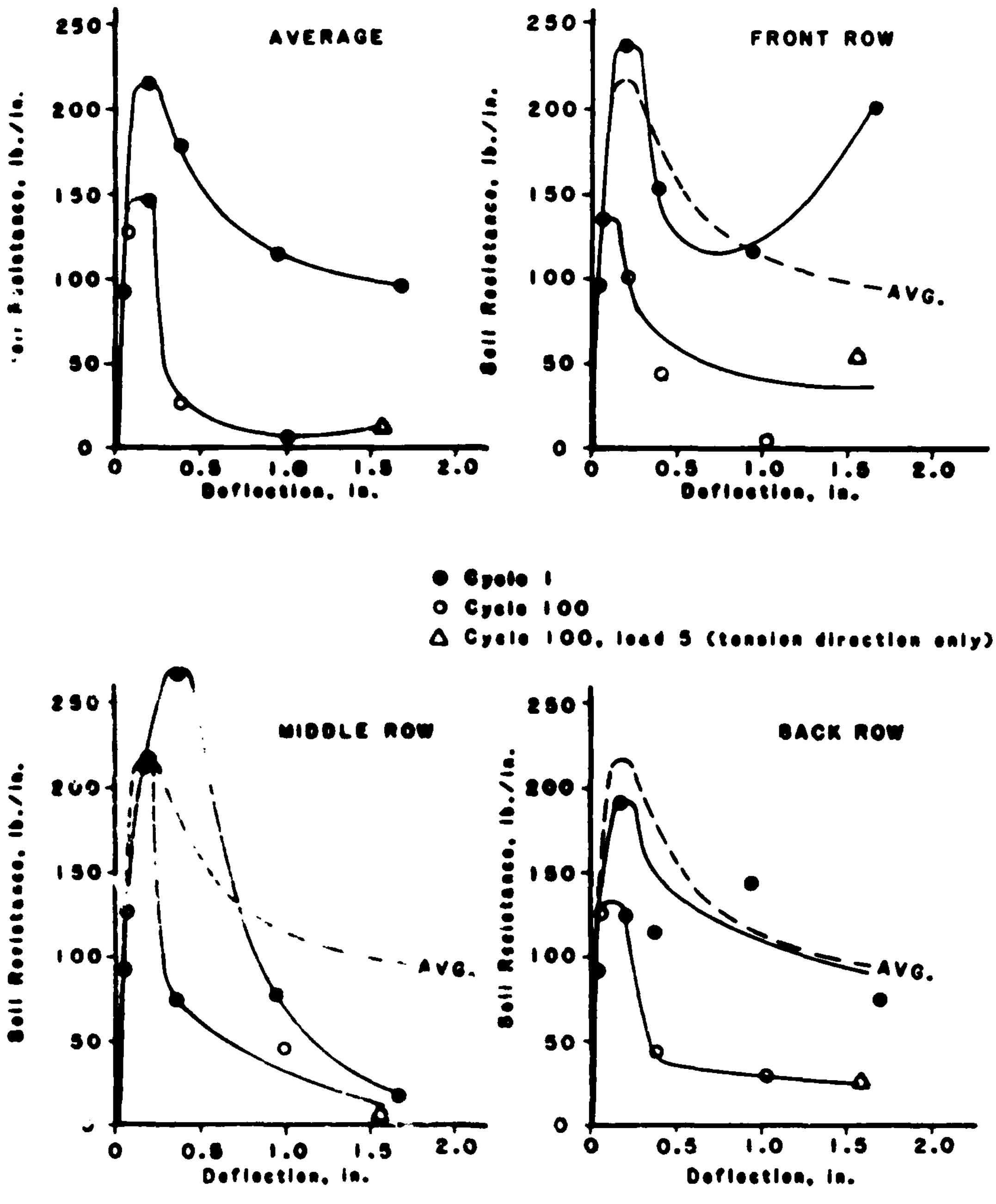


Fig. 7.21 Measured P-Y Curves, 1 Ft. Depth

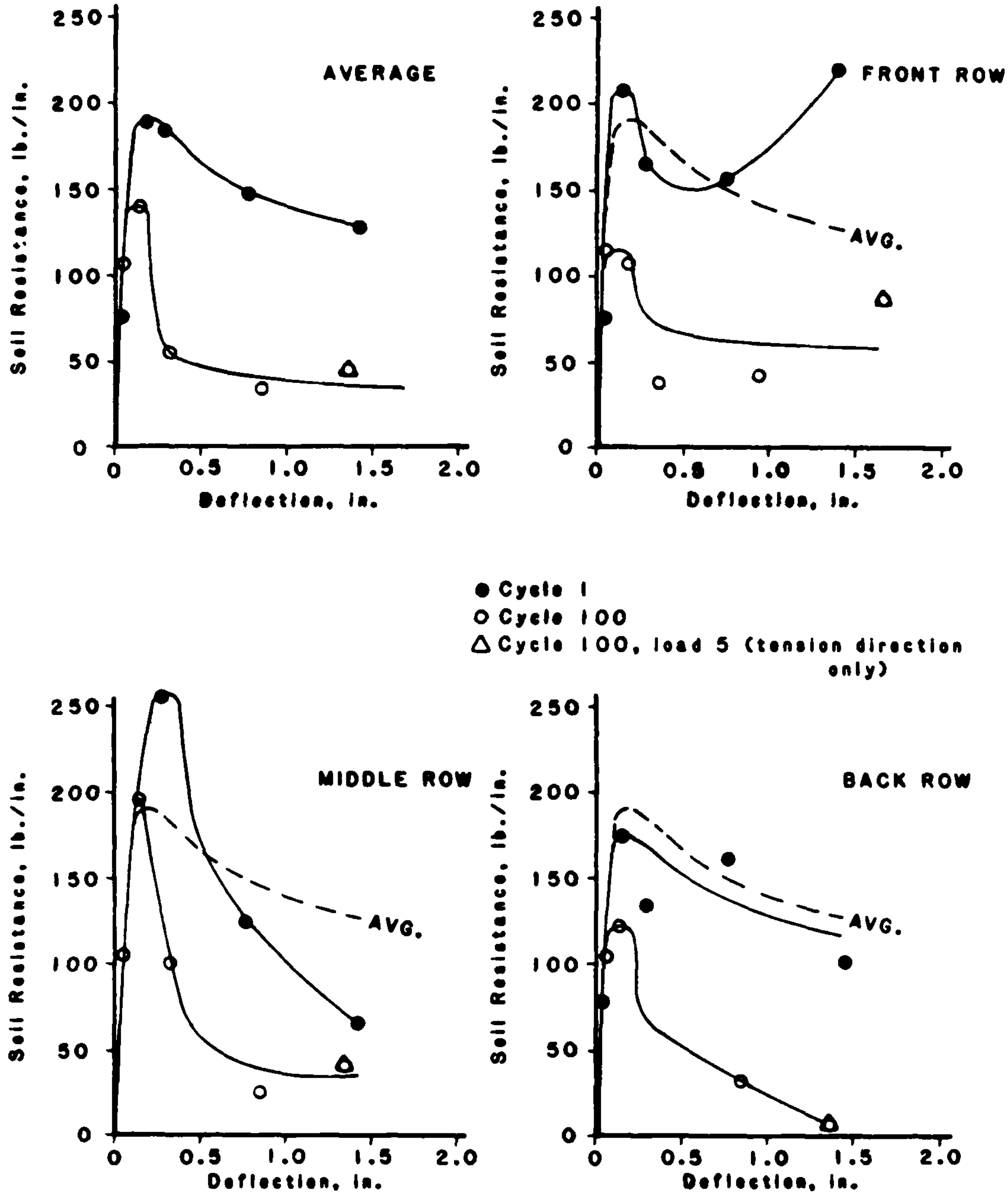


Fig. 7.22 Measured P-Y Curves, 2 Ft. Depth

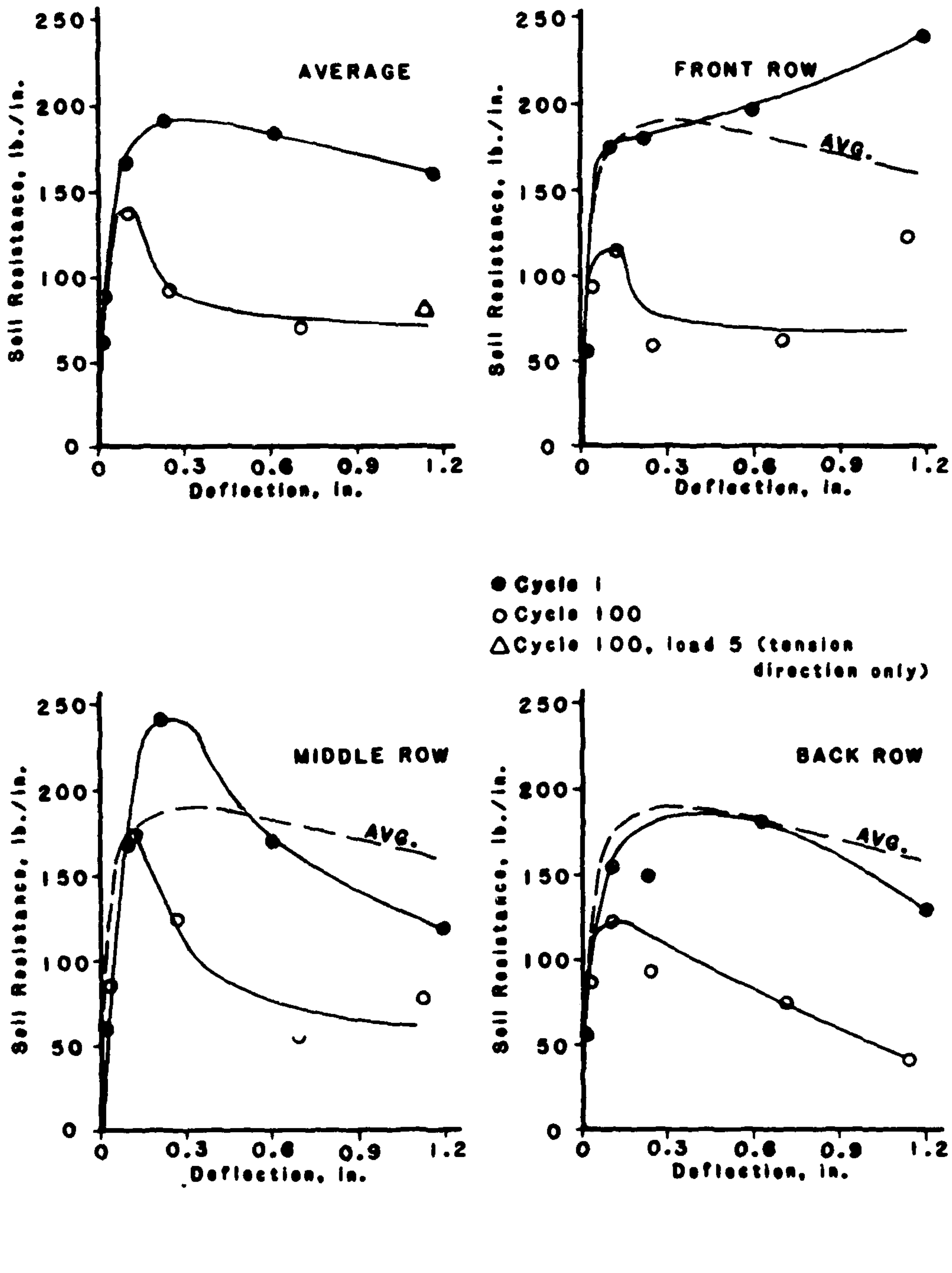


Fig. 7.23 Measured P-Y Curves, 3 Ft. Depth

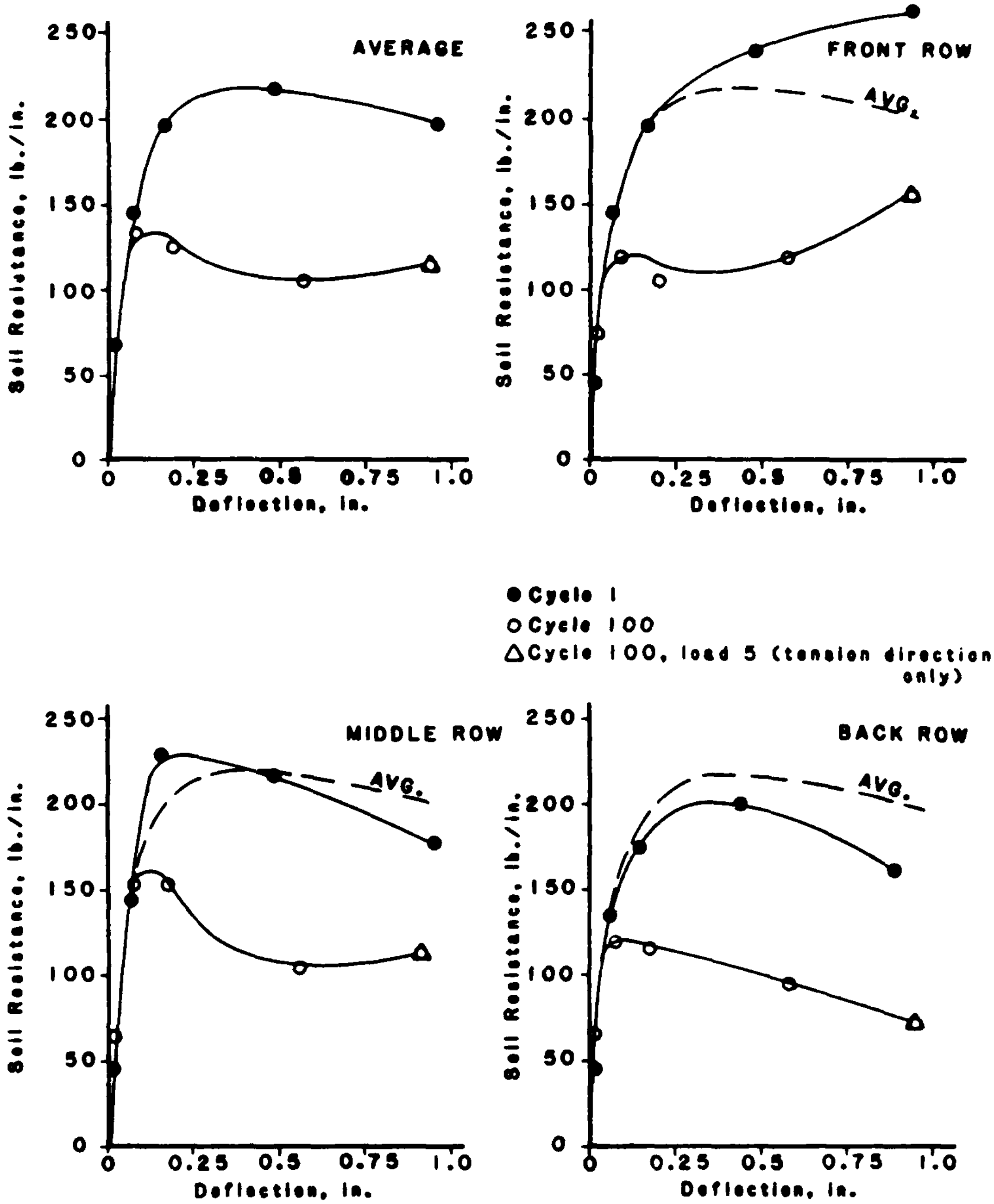


Fig. 7.24 Measured P-Y Curves, 4 Ft. Depth

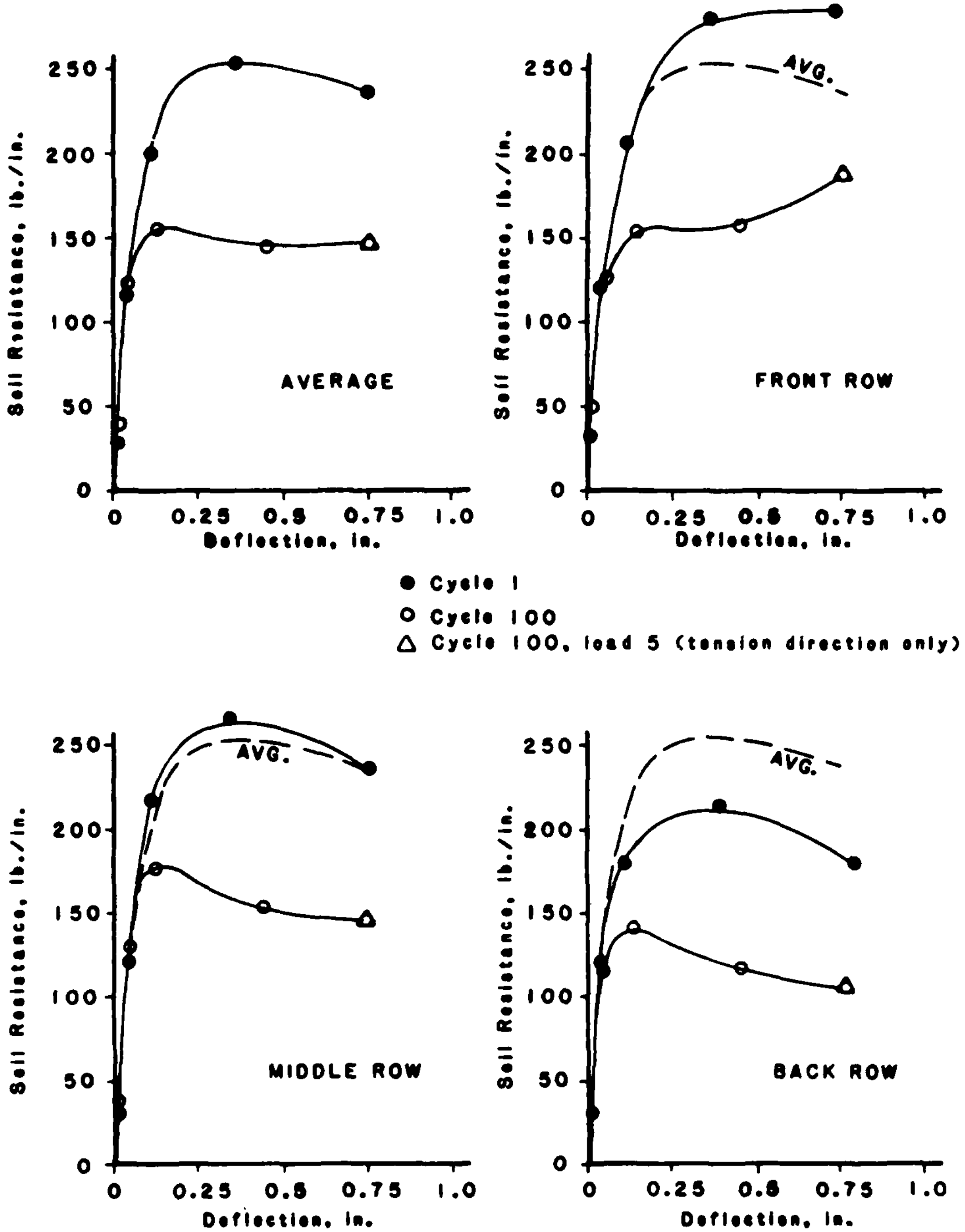


Fig. 7.25 Measured P-Y Curves, 5 Ft. Depth

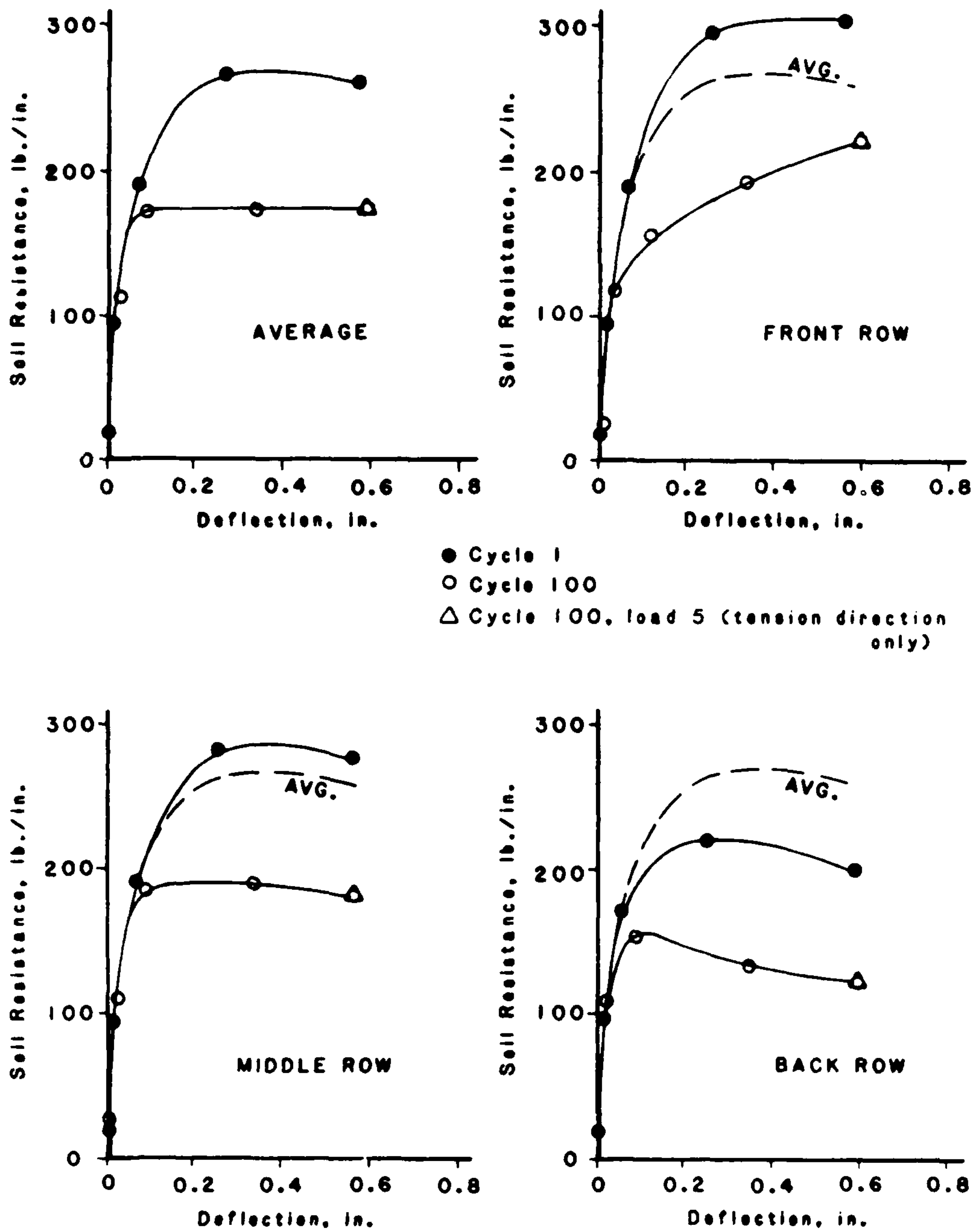


Fig. 7.26 Measured P-Y Curves, 6 Ft. Depth

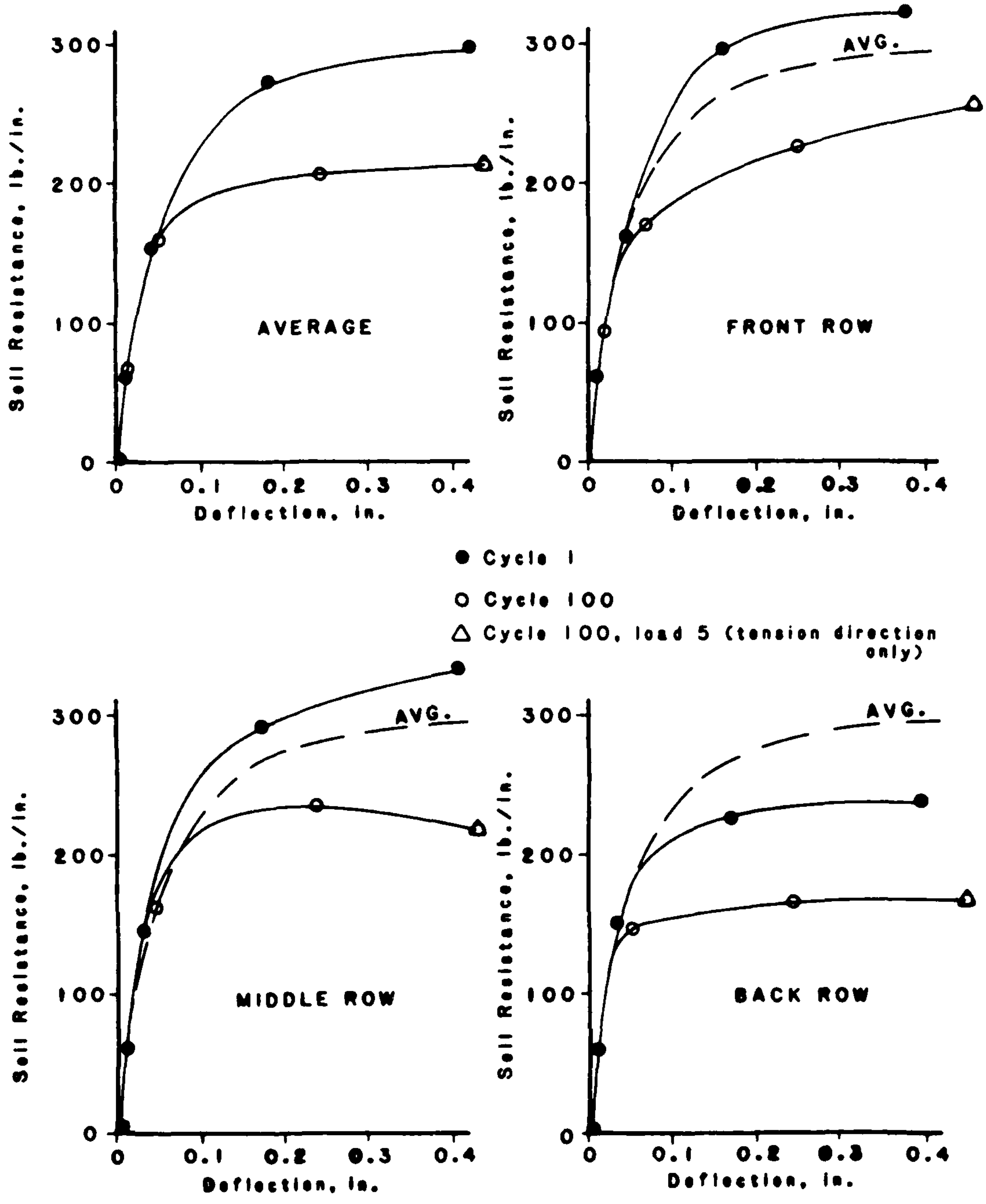


Fig. 7.27 Measured P-Y Curves, 7 Ft. Depth

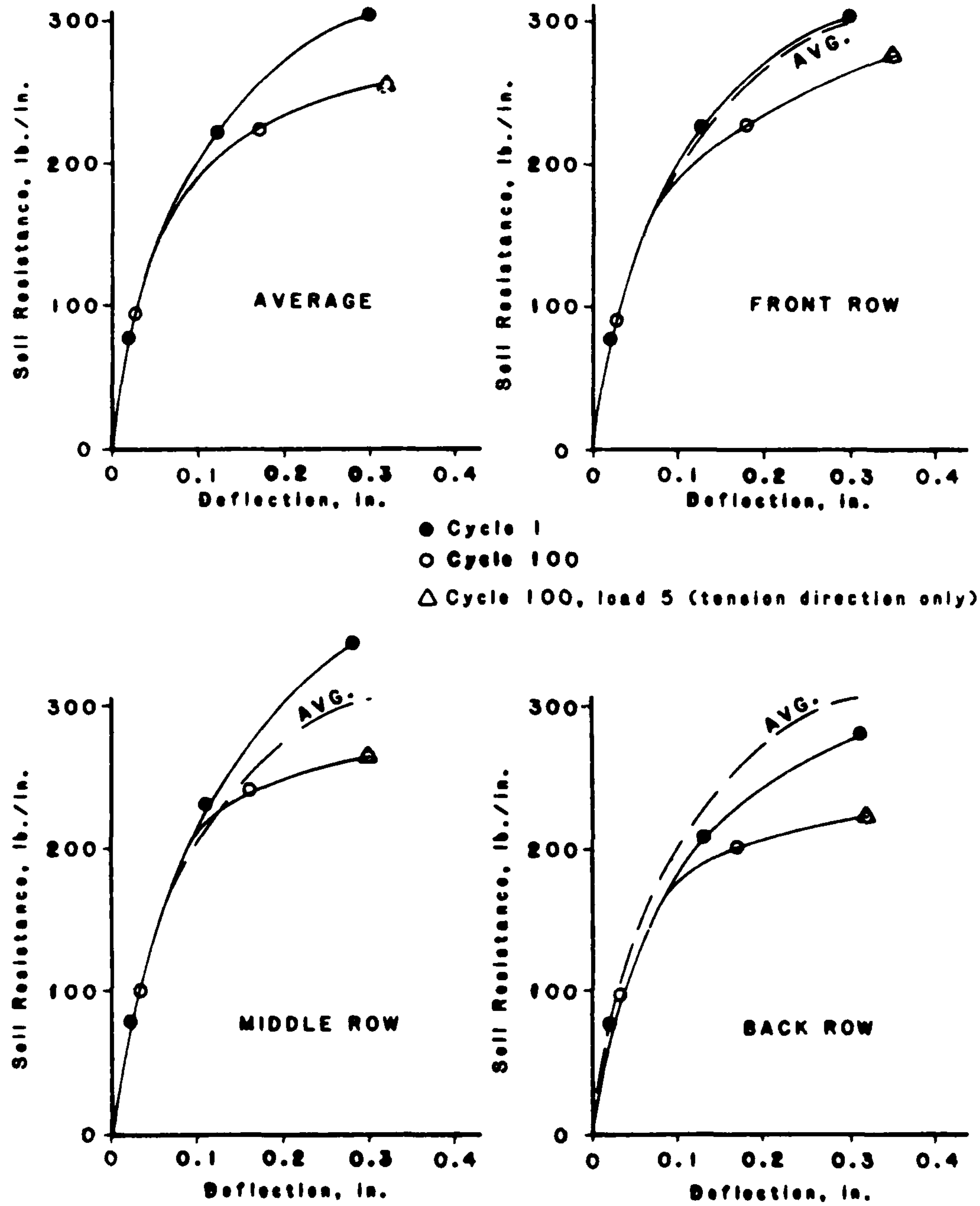


Fig. 7.28 Measured P-Y Curves, 8 Ft. Depth

sion, load 5 was cycled in the tension direction only. As noted earlier, the curves for the 1 and 2 ft depths are considered less reliable than those at lower levels.

Discussion of Results from Cycle 1 (Static)

Some significant results for the measurements by row are evident in the p-y curves for cycle 1. For depths of less than 6 ft the middle row soil resistances were the largest at early loads. At depths of less than 5 ft, the middle-row curves peaked much earlier than the average and subsequently dropped off to a much lower soil resistance at larger deflection. By contrast, the values of soil resistance for the front row were not greatly reduced with increasing deflection, but often continued to increase. Soil resistances for the back row peaked early and diminished, much like the middle-row curves but at a smaller value of soil resistance.

The curves presented in Fig. 7.23 for the 3-ft depth exhibit behavior typical of that described above. Of interest is the relatively sharp peak in the p-y curve for the middle row followed by a reduction in p to less than half of the peak value at a large deflection. The back-row curve was similar in shape but with a less drastic peak and reduction, while the front-row curve did not drop off at all. The average curve indicates decreasing soil resistance at large deflections, as was the case for two-thirds of the piles used to compute the average.

The drop in soil resistance at large deflection for middle- and back-row piles is likely the result of modification of the shear zone of the individual piles by those piles in the foreground. This

"shadowing" effect is not very important at small deflections because the shear strength in the entire shear zone is not fully mobilized. It is also reasonable that a post-peak drop in soil resistance occurred only for shallow p-y curves in general; as soil resistance at greater depths reaches a limiting value and begins to mobilize a zone of shear, some reduction in soil resistance above that depth seems reasonable. Furthermore, the cavities produced by leading piles become larger near the surface.

Perhaps less easily explained is the observaton that the middle row had a higher peak soil resistance at small deflections than did the front row. It might be logical to attribute this phenonmenon to installation effects, in that consolidation due to dissipation of excess pore pressures after pile driving may have resulted in higher horizontal in-situ stresses and increased undrained shear strength for the soil within the group. However, this high peak in soil resistance at small deflection diminishes relative to the front-row soil resistance with increasing depth to a degree that does not seem explainable by installation effects alone.

Another explanation for the unexpected distribution of soil resistance involves the effects of cyclic loading. An observation noted earlier was that loads 2 and 3 did not have a great separation in load, and that the cycling during load 2 could conceivably have affected the cycle 1 results for load 3. With loading in 2 directions, the soil in front of a middle-row pile for "load 3, cycle 1, compression" had just previously been stressed in the opposite direction by the back row piles of "load 2, cycle 200, tension." The

soil ahead of the front row had not been subject to such a large stress reversal during "load 2, cycle 200, tension" and might thus require more deflection to mobilize fully the soil resistance. The consequence of insufficient load separation between loads 1 and 3 might therefore have been more significant for the front-row piles. Of course, if this argument were entirely satisfactory one would expect the back-row p-y curves to also have a significant peak at load 3; such was clearly not the case. This argument might also seem to imply that the middle row would mobilize more soil resistance in the cycle-1-tension measurement than in the cycle-1-compression measurement because of the fact that cycle-1 compression was performed first and the soil ahead of the pile in the tension direction might thus be "preloaded" for mobilization of soil resistance at a smaller strain. Although the plotted points are not direction-dependent (they are averages of both directions), it was quite obvious during data reduction that there was no significant difference in the cycle-1 soil response in the tension and compression directions.

Plots of soil resistance vs. depth for the cycle-1 measurements are summarized in Fig. 7.29. Plots are presented for the maximum (peak) soil resistance as well as for the soil resistance at large deflection. These plots illustrate the trends discussed previously, and show particularly the "shadowing" effect of the front-row and the middle-row piles at large deflections. The greater rate of increase of the large-strain soil resistance for the middle-row piles relative to the front row is of interest. Due to the smaller deflections at greater depths, the middle-row piles are again mobilizing a propor-

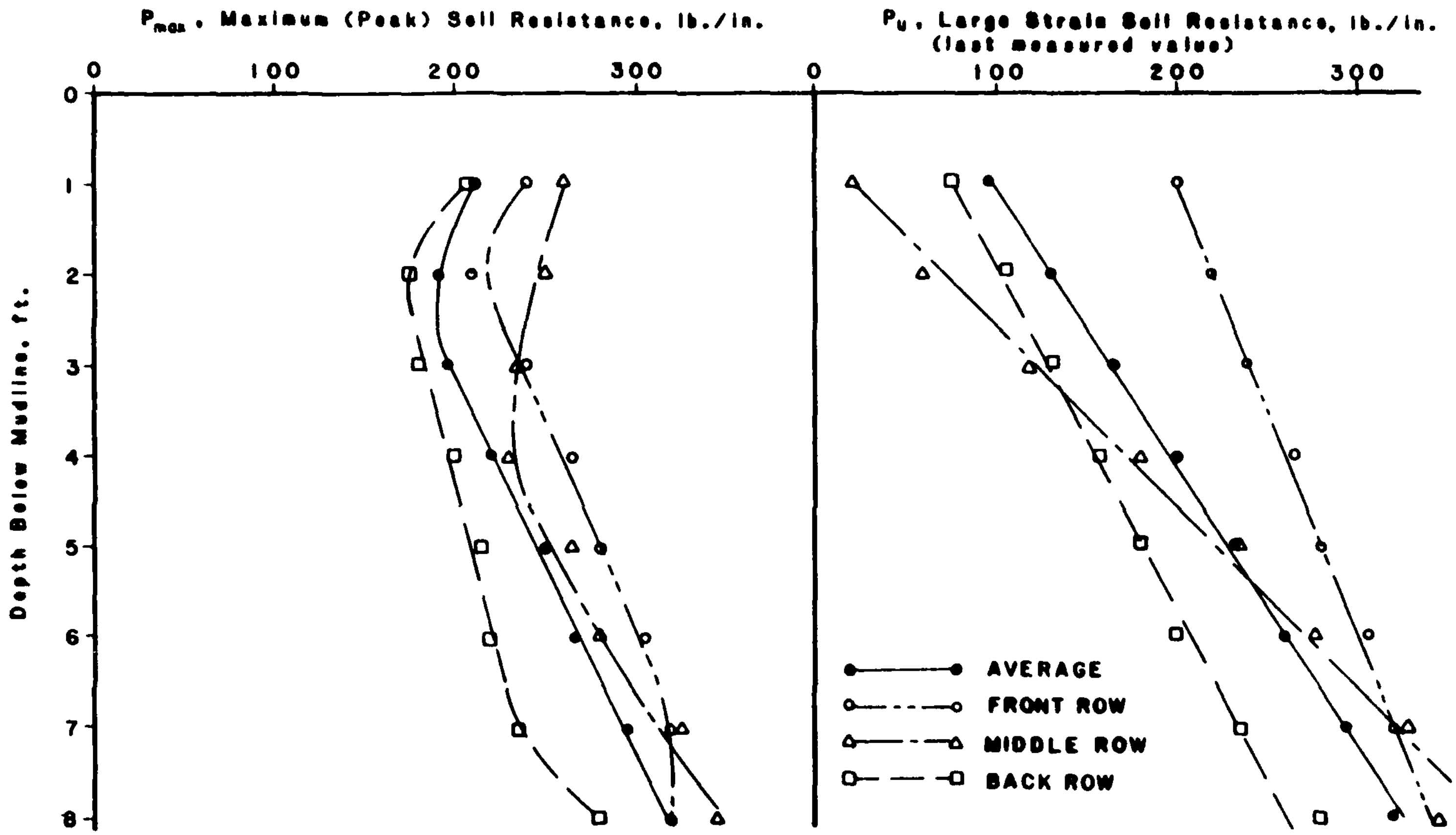


Fig. 7.29 Soil Resistance vs. Depth, Cycle 1

tionally greater soil resistance than are other piles. This result is similar to that noted for measurements at lesser loads at shallower depths.

To provide a comparison of these results with those of the single pile, plots of ultimate (and maximum) soil resistance vs. depth are presented in Fig. 7.30 for the average pile in the group as well as for the single pile.

The difference between the group- and single-pile behavior evidenced by this plot is striking. The figure shows clearly that there exists some reduction in maximum soil resistance due to group effects, and that the effect becomes more significant with depth. Also presented in the figure are plots of single-pile values using the 1980 API Design Rules, which are basically taken from Matlock (1970) as follows.

$$P_u = 3 + \sigma'_z/S_u + J(z/b)$$

where:

σ'_z = effective overburden stress at depth z,

S_u = undrained soil shear strength at depth z,

J = empirical constant with an approximate value of 0.5 for soft offshore clays of the Gulf of Mexico and a value of 0.25 for somewhat stiffer clays (used for this test site), and

b = pile diameter.

The API Design Rules do not agree espcially well with the measured single-pile response but are given as a basis for comparison with the other two curves.

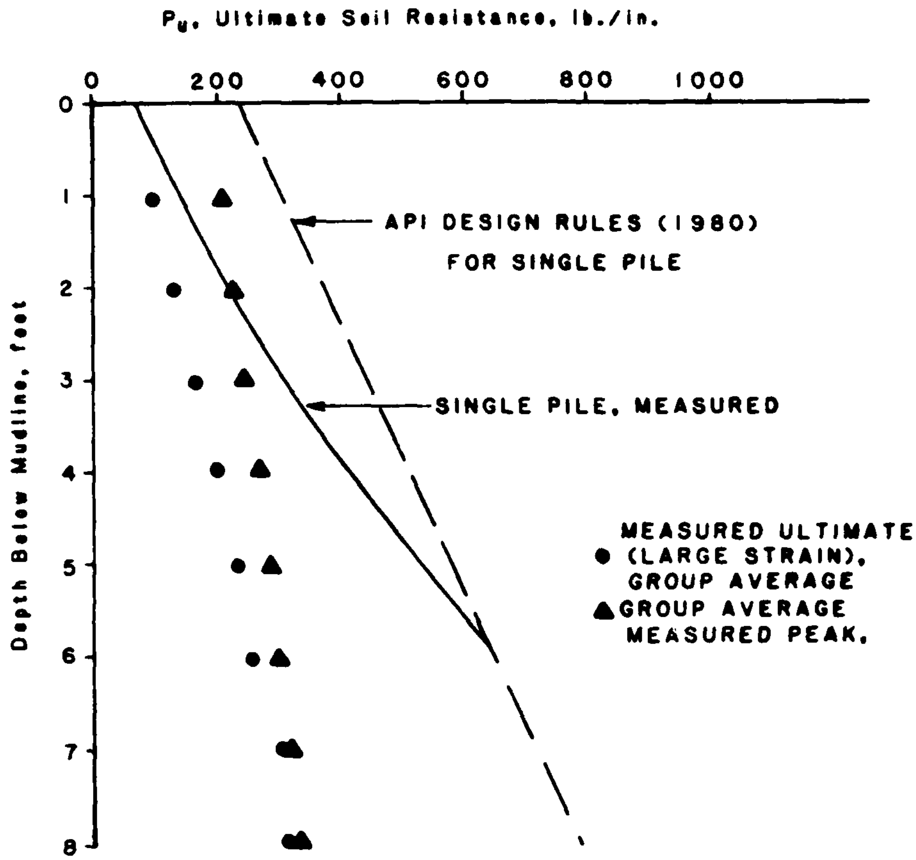


Fig. 7.30 Ultimate Soil Resistance vs. Depth,

Average Pile, Cycle 1 (Static)

Results for Cycle 100 (Cyclic)

Curves showing p vs. y for cycle-100 measurements are shown together with the cycle-1 measurements in Figs. 7.21 through 7.28. Although there exists some scatter in the curves for the top 2 ft (a comment was given earlier about load 3), the general trend is that of a leveling off of the cyclic p-y curves from some point on the static curves. Up to a point of about 50% of the maximum soil resistance for cycle 1, there appears to be little effect of cycling. Figure 7.31 presents a plot as a function of depth of the measured ratios of the "residual" soil resistance (the flat part of the cycle-100 curve) to the ultimate (large strain) cycle-1 value. Also shown are points from the single-pile data. There appears to be little difference in this ratio between the group piles and the single pile, a fact that might at first glance imply that the effects of cycling are not greatly affected by group action. There is a fairly major difference in the top 2 ft, which probably is due to gapping producing shear zone interference. However, the values of ultimate soil resistance for the group piles are greatly reduced as compared to those for the single pile; Fig. 7.31 merely implies that cycling affects the group piles in a similar manner as does the static loading.

Figure 7.32 presents plots as function of depth of peak and residual (large strain) soil resistance for cycle 100 by rows and as an average for the entire group. Comparisons between values for each row indicate trends similar to those observed for cycle 1. Peak values tend to be high for middle-row piles, while residual (large strain) values are largest in the front row.

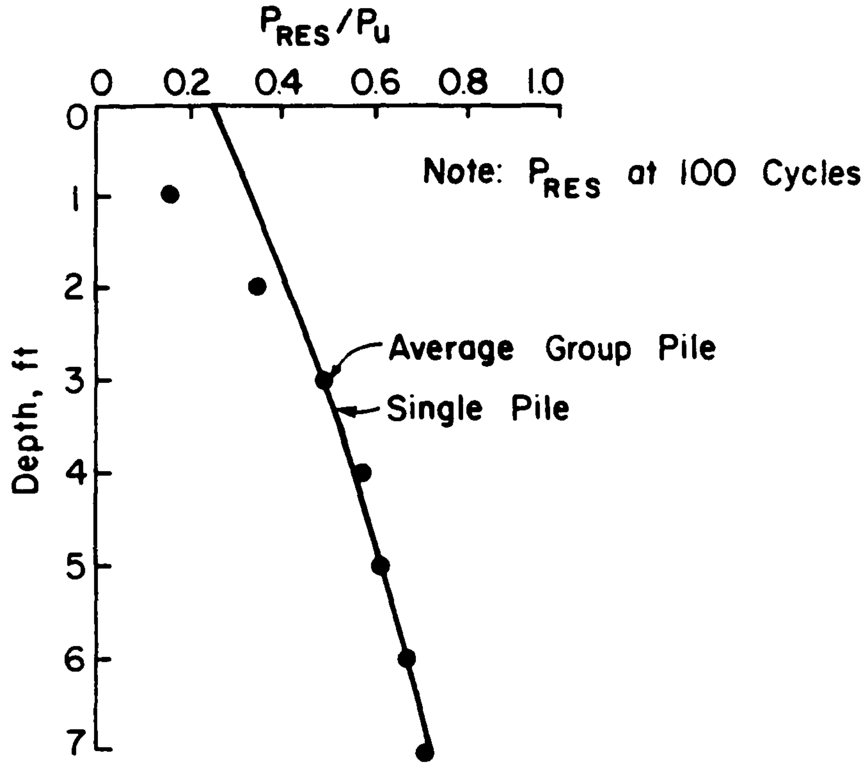


Fig. 7.31 Ratio P_{Res} / P_u vs. Depth

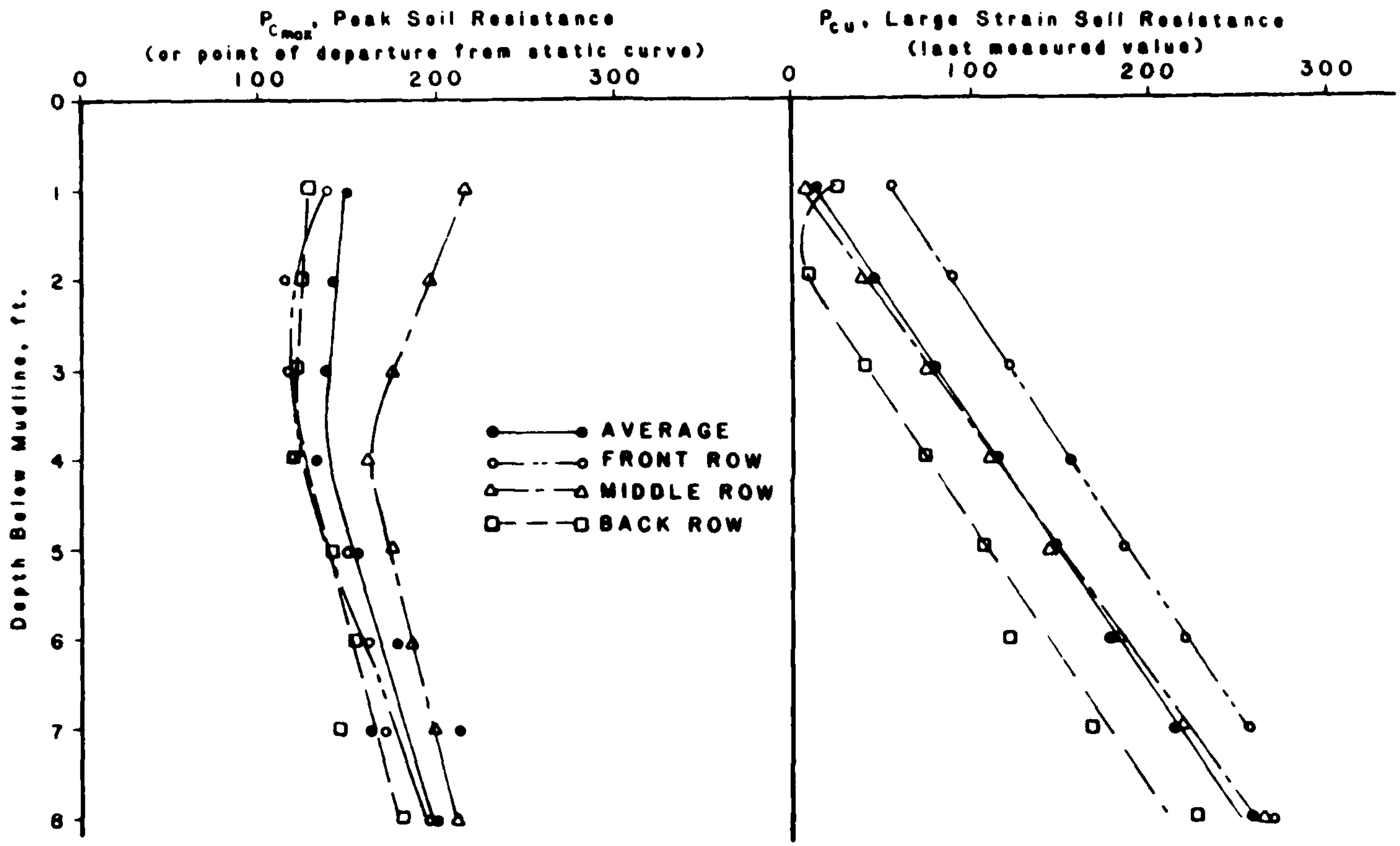


Fig. 7.32 Soil Resistance vs. Depth, Cycle 100

SUMMARY

Load-test data are presented in this chapter in a manner intended to highlight some of the significant aspects of the pile-group behavior as well as to contrast this behavior with that of a single pile of similar size and under similar loading conditions.

It appears that the differences in behavior between the piles in the group were significant. The deflection under load of the piles in the group is significantly greater than that of a single pile under a load equal to the average load per pile. The bending moments in the piles in the group are greater than that for the single pile and the maximum moments are shifted deeper. The maximum soil resistance for the piles in the group is greatly reduced as compared to the single pile, and this reduction is more significant with depth. The greatest portion of the load on the group is distributed to the piles in the front row, diminishing to the back row. Variation of load in the piles in the group was generally 20% or less about the average, and the variation of maximum bending moments was somewhat less. The behavior of the piles in the group will be examined in the following chapter in more detail, as the measured results are contrasted with the predictions from the use of several models that are typical of those in current practice in design.

CHAPTER 8

COMPARISON OF RESULTS FROM EXPERIMENT WITH RESULTS FROM SEVERAL METHODS OF ANALYSIS

INTRODUCTION

Presented in this chapter are the results of analyses of the pile-group response using the analytical models described in Chapter 4. These analytical models are thought to represent the current state-of-the-art in the practice of geotechnical engineering. The predictions have been made using the properties of the soil as described in Chapter 6 as well as the results of the test of the single pile that was instrumented. These data on soil properties and pile testing exceed in quantity and quality that which is normally available on a typical design project. One might therefore conclude that the predictions of these models represent the minimum error to be expected on a real project. All of the analytical models in use today have some degree of empiricism, at least as related to predicting the response of an isolated, generic pile to lateral load. By making use of the results from the load test of the single pile to "back fit" pertinent inputs to the analyses, the errors in prediction should be largely due to errors in modelling group action rather than due to errors in modelling the behavior of single laterally-loaded piles in general.

The order in which these results are presented departs somewhat from the order of presentation in Chapter 4. Because the hybrid

models of Focht and Koch and of PILGP2R are based upon the earlier elasticity approach of Poulos (which ultimately evolved to DEFPIG), the use of these models is presented following the use of DEFPIG. Following the hybrid-model results are the predictions from the single-pile representation and the Bogard-Matlock procedure.

DEFPIG

DEFPIG is a computer program to obtain a solution using the approach detailed in Poulos' well known 1971 papers (May, 1971); however, the computer program has a few features not described in those articles. DEFPIG has been used to analyze conveniently the pile group as might be done using the charts in the 1971 papers. The added features, allowing local yield in the soil and a variable elastic modulus with depth, have permitted a more sophisticated analysis when using the elasticity-based model. Although these added features violate some of the original assumptions of the method, with proper calibration to experimental results and with the use of appropriate engineering judgement, this model may be an efficient analytical tool for designers. The usefulness of the code DEFPIG is limited in comparison with some other models in that the code used in this study has no provisions for computing the response to cyclic load or for predicting the distribution of bending moments with depth. It does predict the load vs. deflection at the pilehead as well as the distribution of load to the individual piles. DEFPIG also provides a convenient first step to establishing the elasticity-based parameters used in the Focht-Koch hybrid model.

Predictions Using a Soil Modulus that is Elastic and Constant with Depth, No Local Yield

As a first step in the analysis, equivalent values of modulus for a homogeneous elastic soil were backfigured based on the results of the single-pile test. It has long been recognized that the deflection of laterally loaded piles at working loads include a considerable amount of inelastic deflection (Poulos, 1975 and Focht-Koch, 1973). When compared with experimental results from a full-sized load test, the results from computations assuming elastic response may provide insight into the errors associated with the assumption of elastic response. The results of the computations also may indicate the magnitude of load to which the assumption of elastic response can be reliably used.

Presented in Fig. 8.1 is a plot of load vs. deflection that results from the use of computer program DEFPIG with a soil modulus that is elastic and constant with depth and with no local yield. The results from the test of the single pile are shown, along with the straight-line predictions for a single pile in elastic soil. For each different value of soil modulus, the load was found at which the elastic analysis correctly predicted the deflection of the single pile. This load was then used as the average load per pile in an elastic analysis for the group. Each value of soil modulus thus produced one point on the load-deflection plot for the group, at a load for which the soil modulus correctly predicted the deflection of the single pile. The computed points are shown in Fig. 8.1. It appears that the procedure overpredicts the group deflections by a factor of about 2

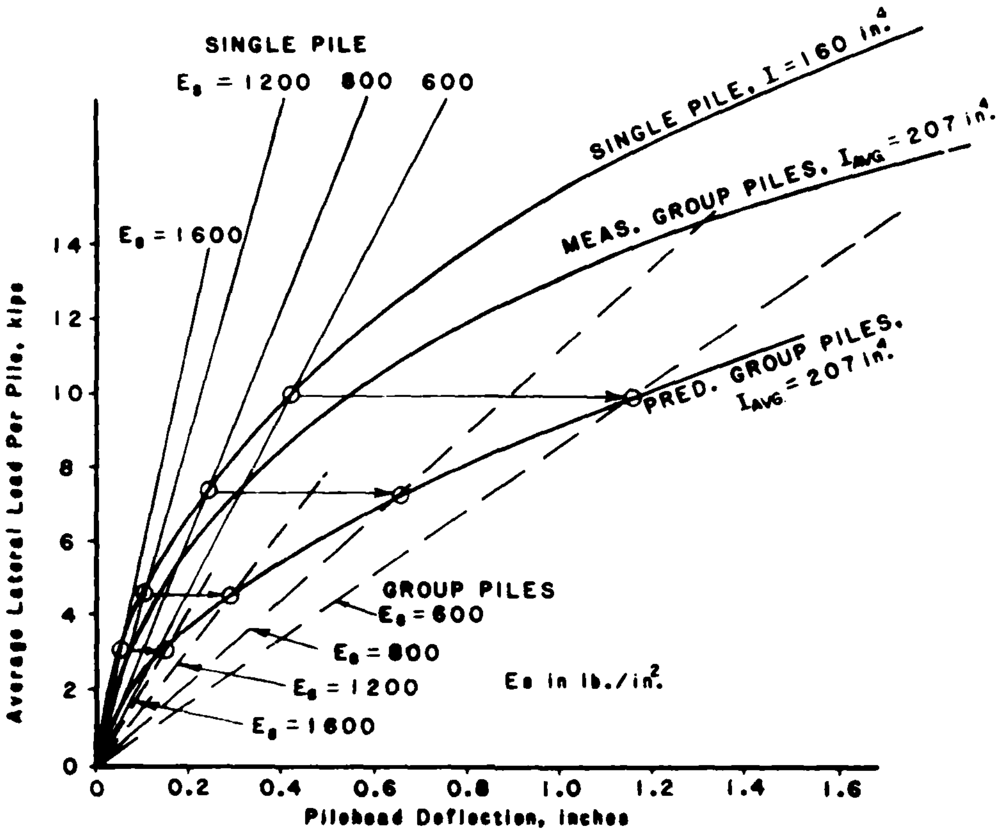


Fig. 8.1 DEFPIG Lateral Load-Deflection Predictions at Working Loads Using Constant E_s, Fitted to Single Pile Data

throughout the range of loading that is analyzed. Regardless of the value of the soil modulus that is used, for this particular configuration of the group of piles, the deflection of the group is always 2.8 times the deflection of the single pile at the same load per pile. The plots of experimental results appear to indicate that very little, if any, pile-soil-pile interaction occurred at small loads because the deflections of the single pile and of a pile in the group are almost the same.

It may be noted that the analysis described above could have been performed using equivalent deflections for the single pile and group rather than load. A predicted curve for the response of the group would be obtained by projecting the points fitted to the single-pile data on Fig. 8.1 vertically (at a constant deflection) to obtain the average load in the group piles at that deflection and modulus. A more conservative (and more erroneous) prediction of group response would result.

Predictions Using a Soil Modulus that Varies with Depth and Using Local Yield

With an analysis which includes local yield, the soil modulus for the elastic range should correspond to small strains. The use of a value of soil modulus of (E_s) 400 to 800 times the undrained shear strength is thought to be representative of the elastic range. There exists virtually no experimental guidance for designers in selecting limiting values of soil pressure that correspond to local yield, other than those derived from load tests of single piles that are analyzed using subgrade-reaction theory. Local yield is undoubtedly the most

influential factor in the analysis using DEFPIG. For the computations in this section, the soil pressures where local yield will occur have been taken from the results of the load test of the single pile as equal to the maximum soil pressures from the load-transfer curves.

Presented in Fig. 8.2 are load-deflection results from DEFPIG compared with both the single-pile and the group-pile-test results. Good agreement between experiment and analysis for the single pile is evident and expected because the results of this test were used to establish the soil pressures for local yield. For the pile group, DEFPIG significantly overpredicts deflection throughout the working-load range for both the $800S_u$ and the $400S_u$ assumptions for E_s. Results are also shown for an elastic soil modulus of 30,000 lb/sq in.,which is a small-strain modulus based on shear-wave measurements using the crosshole technique. Even with this small-strain modulus, DEFPIG significantly overpredicts deflection. As a comparison, a plot is presented where a value of E_s of $800S_u$ is employed and with the assumption that there is no pile-soil-pile interaction. The plot appears to indicate that the pile-soil-pile interaction is overestimated when using DEFPIG with the modifications that are indicated.

There is an indication that the values of local yield for a pile group might need to be reduced from those of a single pile due to group effects. A reduction of 78% in local yield for the group was used to produce another plot in Fig. 8.2. The 78% value was simply estimated based on the ratio of the frontal exposure of the group divided by the sum of the pile diameters (7 diameters/9 diameters). Obviously, if the deflections were overpredicted without using a

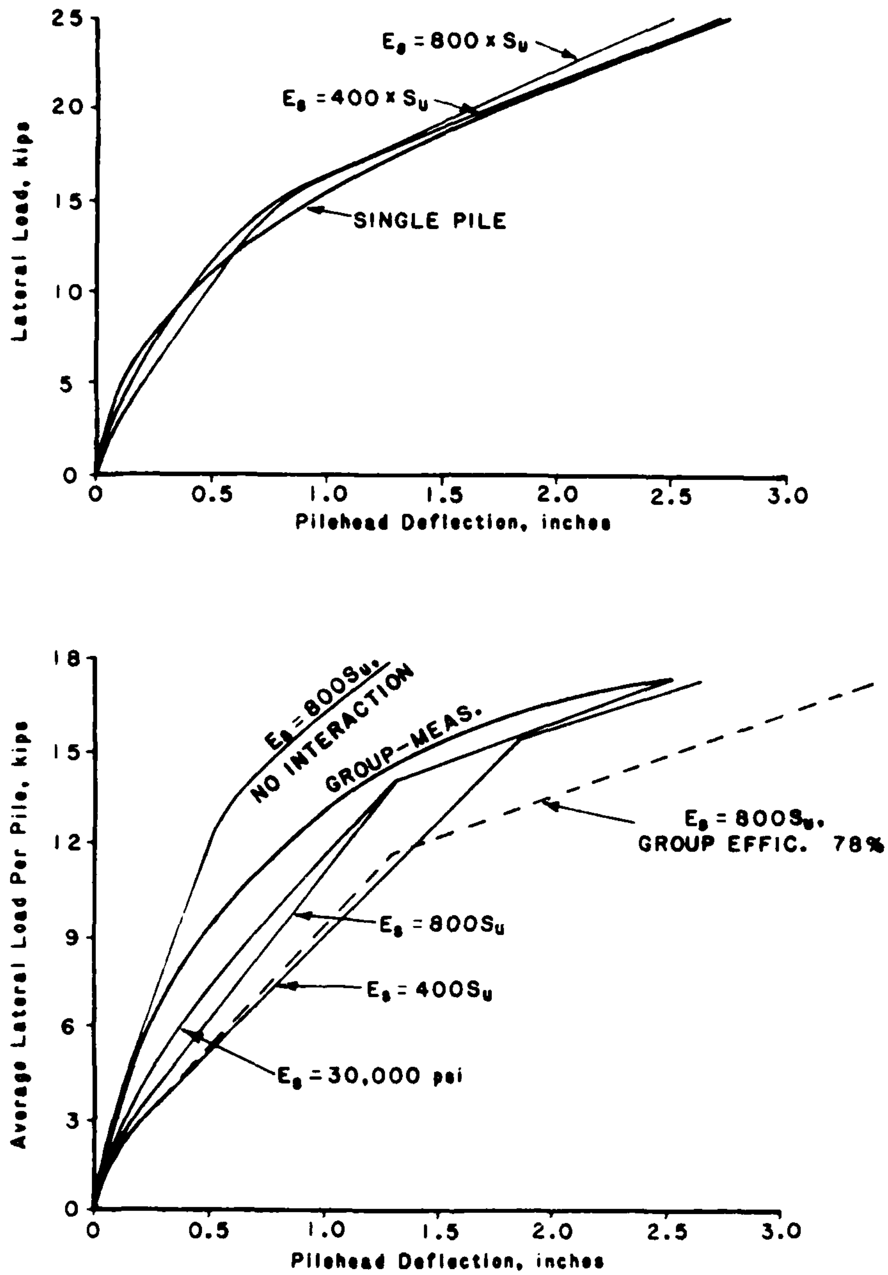


Fig 8.2 DEFPIG Lateral Load-Deflection Predictions Including Local Yield

reduction in local yield for the group, this additional factor does not improve the prediction.

Interaction Factors

Because the two-pile-interaction factors are used subsequently in the Focht-Koch procedure, the results of the DEFPIG analyses were used to examine the influence of the assumptions regarding E_s on the interaction factors. Unless specified as input, DEFPIG computes interaction factors. Computed interaction factors for the analyses discussed previously are presented in Fig. 8.3. The plots for E_s equal to $400S_u$ or $800S_u$ produce virtually the same line and are indistinguishable. The use of a constant E_s produces slightly higher interaction factors and could thus be considered to be conservative. These factors are computed for two piles embedded in elastic soil, and the effects of local yield on the distribution of soil reaction and pile-soil-pile interaction are not considered.

Distribution of Load in the Group

DEFPIG computes the distribution of load to the piles in the group using the two-pile-interaction factors. Results of computations using E_s equal to $800S_u$ are presented in Fig. 8.4. In the DEFPIG procedure, deflections of a single pile are computed considering local yield and the group deflection is computed to be some factor times the single-pile deflection. The effect of the variation of load in the piles in the group and any possible variation of local yield within the group is not considered. The prediction of the distribution of load using DEFPIG is the same regardless of the assumptions concerning local yield. For an actual case, one would expect that the piles that

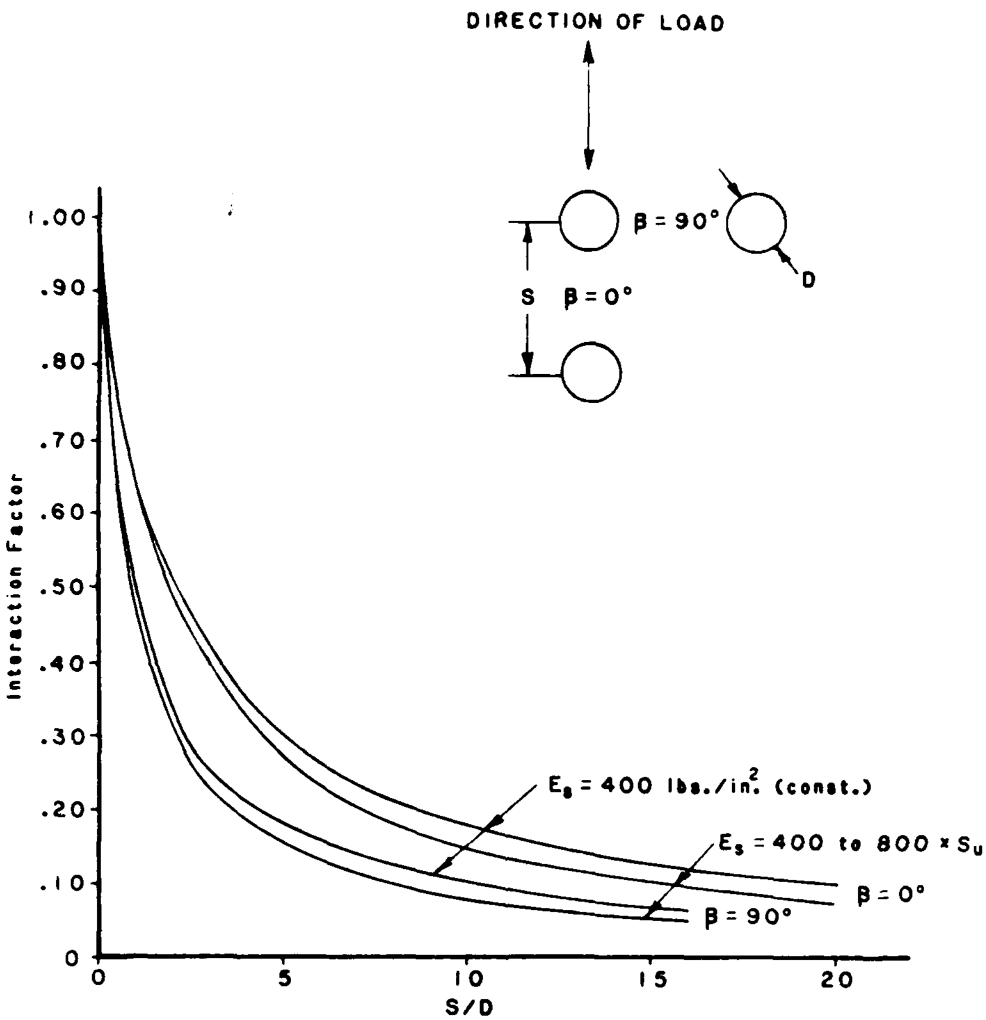


Fig. 8.3 Effect of E_s on 2-Pile Interaction Factors

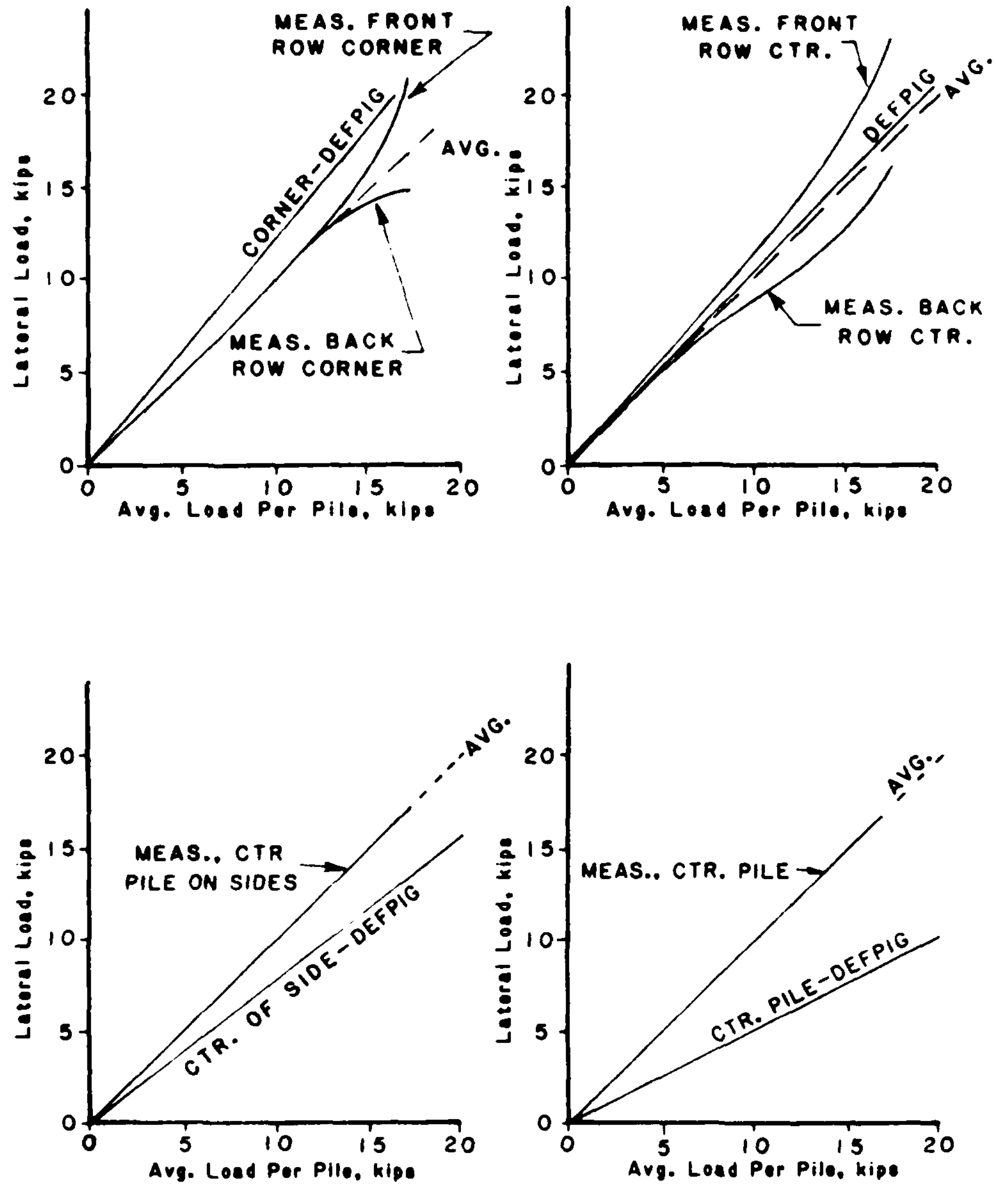


Fig. 8.4 Distribution of Load by Location, Cycle 1; DEFPIG Prediction vs. Measured

are more heavily loaded within the group would shed load to piles that are less heavily loaded. A more rigorous analysis of the distribution of load to individual piles would thus be expected to result in a more even distribution of load. As evident from Fig. 8.4, DEFPIG predicts a greater variation in load among the group piles than was observed. Loads to corner piles are overpredicted, especially to those in the back row, and loads to other piles are underpredicted. The loads in the piles in this elastic model are symmetric, with the same load in the back row piles as in the front row. The observed loads on the piles were significantly biased toward the front row.

Summary

DEFPIG is an elasticity-based model with added provisions to account for local yield of the soil and a soil modulus that varies with depth. DEFPIG appears to overestimate pile-soil-pile interaction for the experiment that was performed and thus overpredicts deflections of the group. Local yield of the soil is very influential in computing pile response and there is at present little guidance to allow designers a rational selection of local-yield parameters. Distribution of load to the piles is computed on a purely elastic basis and appears to overpredict variations within the group. In particular, DEFPIG predicts a symmetric pattern of distribution of load from front to back that did not agree with observations. The usefulness of DEFPIG for design is limited by the inability at present to consider cyclic loads directly or to output distribution of stresses with depth in the piles.

FOCHT-KOCH METHOD

After the basic procedure for computing the pile-soil-pile interaction in an elastic soil was established by Poulos, Focht and Koch combined the Poulos procedure with existing load-transfer models to allow consideration of nonlinearity in soil response as well as predictions of pile stresses with depth. The Focht-Koch method as proposed in their 1973 paper and as outlined in Chapter 4 of this report has been used to predict the response of the pile group used in this study for both static and cyclic loading. In using the procedure, the results of the test of the single pile were utilized to establish important parameters for both the elastic interaction and the local load-transfer portions of the analysis. Focht and Koch suggest that a value of Young's modulus, E, "probably can be equal to the initial-tangent modulus indicated by most laboratory tests" in order to represent a low level of stress in the soil. Based on the results presented in Fig. 8.1, a uniform value of 2000 lb/sq in. for E_s appears to match the data from the test of the single pile at small loads. This value, as well as a modulus equal to $800S_u$, has been used for the elastic interaction portion of the analysis. The two-pile-interaction factors as computed using DEFPIG are somewhat smaller for the case of increasing modulus with depth. For the p-y portion of the analysis, results of the single-pile test have been used to produce p-y curves for the Focht-Koch analysis. As mentioned in the introduction to this chapter, the use of single-pile-test results implies that differences in the predicted response of the group as compared to the measured response should be attributable largely to errors or miscon-

ceptions in the analytical procedure rather than in determination of material properties.

Static Case

Load-Deflection Predictions. Presented in Fig. 8.5 are the results of analyses using both of the values of E_s as noted above. The deformation predictions for the group appear to be constant multiples of the deformation for the single-pile; such behavior might be expected, because of the fact that the pile-soil-pile interaction is contributing only some added deflection in the analytical algorithm rather than altering the fundamental response of the group. Also, the predictions that are shown represent the solution of the simultaneous equations involving deflection and loads in the piles; the use of y-multipliers and p-multipliers has not entered into the analysis at this point.

The group effects are overpredicted by the analysis at loads less than about 7 or 8 kips per pile. At loads less than about 4 kips per pile, interaction might be expected to be predicted by elastic methods using "small-strain" values of modulus; however, little or no group effects were observed. Of course, precise comparisons of small deflections in the single pile are especially subject to experimental errors. At large lateral loads, group effects were much more significant than predicted by elastic interaction. Obviously, the group effects were highly nonlinear.

Load-Moment Predictions. Presented in Fig. 8.6 are the predictions of load vs. maximum moment for the case of E_s = 2000 lb/sq inch. In computing values for the curves, various y-multipliers were

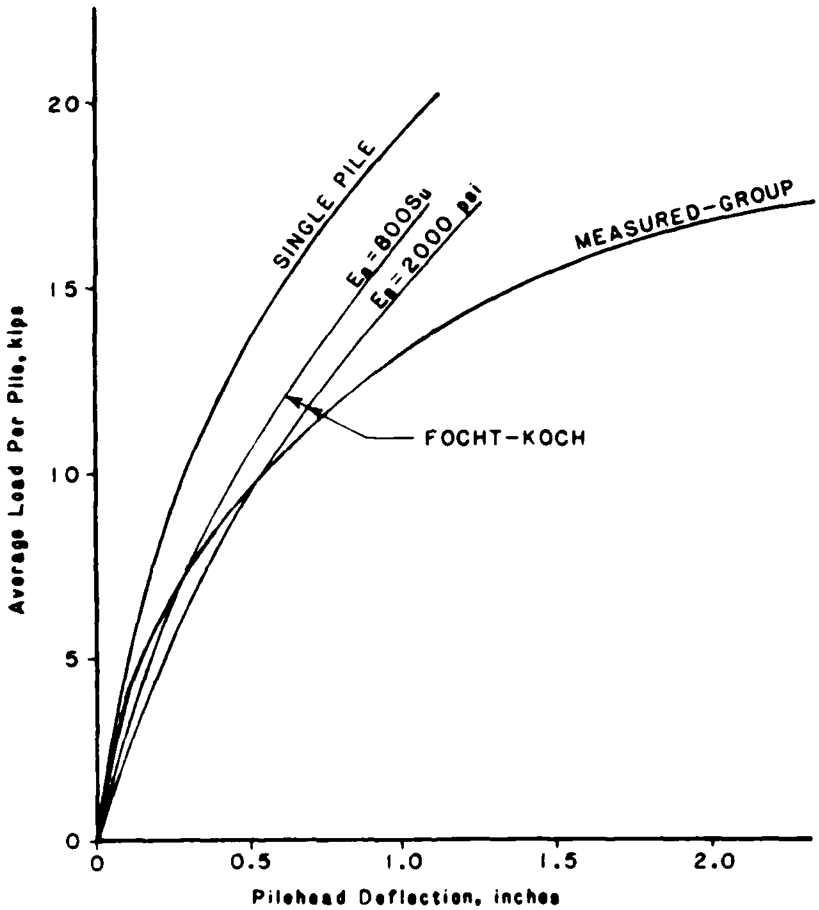


Fig. 8.5 Focht-Koch Lateral Load-Deflection Predictions, Cycle 1 (Static)

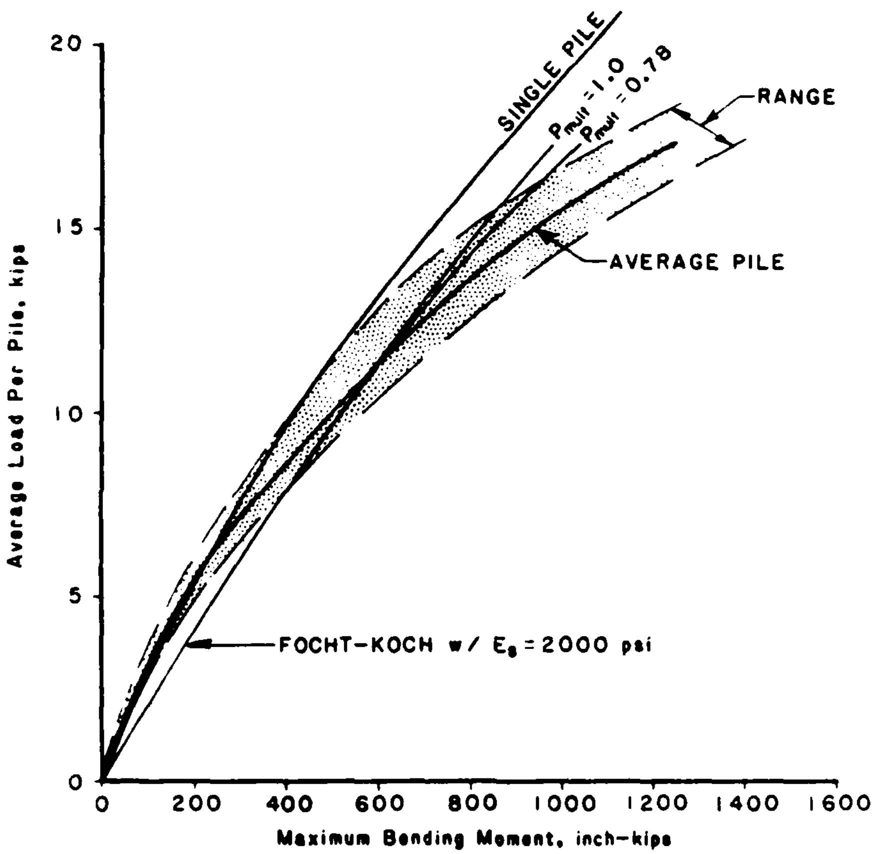


Fig. 8.6 Focht-Koch Load vs. Maximum Moment Predictions, Cycle 1 (Static)

used to "stretch" the single-pile p-y curves such that the pilehead deflection of a single pile would be equal to that from the analysis of the group as presented in Fig. 8.5. The resulting p-y curves were used with the well known COM624 computer program (Reese, 1977) to produce the analytical results. The single-pile load used in the procedure is taken as the maximum load that was computed for any pile in the group. Predictions of distribution of load to the piles within the group result from the solution of the simultaneous equations mentioned previously. In keeping with Focht and Koch's suggestion that a p-multiplier might be used to reduce the values of ultimate soil resistance for group effects, analyses were performed using a p-multiplier of 0.78 as well as 1.00. The 0.78 value was the result of dividing the area of the frontal exposure (7 pile diameters) by the sum of the pile diameters (9 pile diameters).

As was the case for the prediction of load vs. deflection, the Focht-Koch method overpredicted the group effect on moments for small loads, and underpredicted moments for large loads. As is evident in Fig. 8.6, the use of a p-multiplier less than 1.0 did not make much difference; the resulting y-multiplier required to match the pilehead deflection computed earlier simply came out smaller.

The distribution of bending moment with depth is presented for several loads in Fig. 8.7. Again, it is apparent that moments are overpredicted for small loads and underpredicted for large loads. Also notable is the fact that the depth to the point of maximum moment is predicted in a similar manner. The Focht-Koch prediction indicates that the depth to maximum moment does not vary significantly as loads

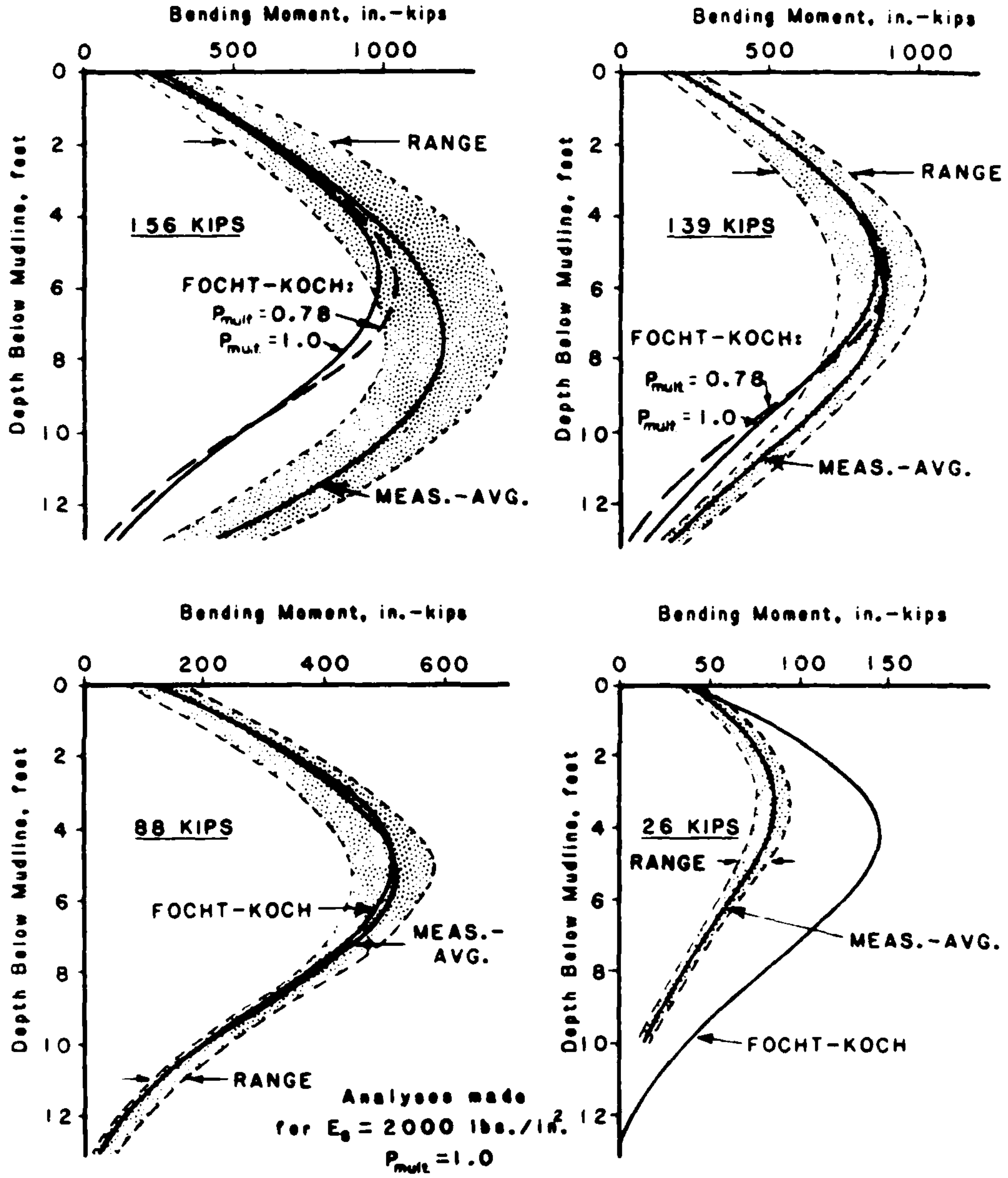
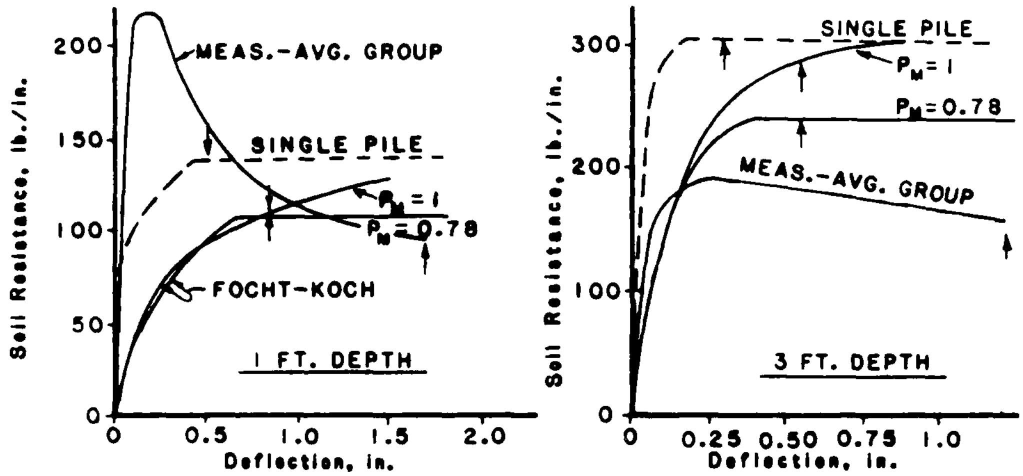


Fig. 8.7 Focht-Koch Bending Moment vs. Depth Predictions, Cycle 1 (Static)

increase, while observations showed that the depth to maximum moment was substantially deeper at larger loads. Again, in the computation of bending moment the analytical results were little affected by the use of a p-multiplier.

Predictions of p-y Curves. Presented in Fig. 8.8 are the p-y or load-transfer curves for several depths that were predicted by the Focht-Koch procedure along with those that were derived from measurements. The curves based on the Focht-Koch method must be constructed point by point because the y-multiplier used to match the group load-deflection predictions varies with load. The curves that are shown are therefore not used in a general sense, but represent a composite of the different curves used in the different analyses. The arrows indicate the point on each curve that corresponds to the last load that was considered; extensions of the Focht-Koch curves beyond that point indicate the curves for the final load. The single-pile curve for loads larger than 17.3 kips is the result of measurements at greater loads and is not simply an extrapolation.

The most obvious and significant feature revealed by Fig. 8.8 is that the p-y curves derived from measurements on the group indicate substantially reduced values of ultimate soil resistance, and that the reduction is not reflected in the predicted curves. This fact undoubtedly accounts for the error in the prediction of the depth to maximum moment, because the reduction in ultimate soil resistance is more significant with depth. The group effect that is predicted for very shallow curves accounts for the overprediction of group effects



NOTE: ↑ marker represents deflection at 17.3 kip average pile load (156 kip group load). Analyses based on $E_s = 2000$ lbs./in.^{2}

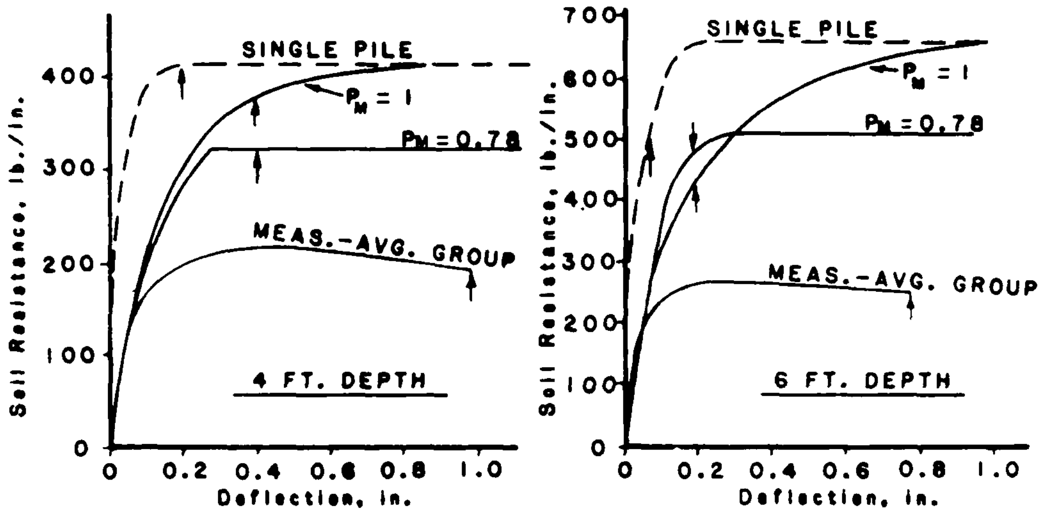


Fig. 8.8 Focht-Koch P-Y Curve Predictions, Cycle 1 (Static)

for small loads. Measurements indicate little effect of adjacent piles on the p-y curve at the 1-ft depth.

Distribution of Load in the Group. The simultaneous equations to be solved in the Focht-Koch method include a weighting factor, R, that is applied to the pile for which a particular equation is written (see Chapter 4 for a detailed presentation of the equations). The result of the use of the R factor is to distribute load more evenly to the piles in the group as compared to the elastic solution. Figure 8.9 illustrates this fact. The pattern of distribution of load is the same as that for the elastic solution (i.e., the corner piles carry the most load, the center pile the least), but the magnitude of the differences is much less for the Focht-Koch method than for the elastic method.

Cyclic Case

The most important effect of cyclic loading on soil response is thought to be the gapping and scour that was observed immediately adjacent to individual piles. Scour apparently accelerated gap formation, with the build up of pore pressure and the "softening" of the soil of secondary importance. The loss of soil resistance due to cyclic loading has traditionally been accounted for by the use of strain softening p-y curves, in which the curve drops to some lesser residual value at deflections greater than that associated with a peak soil resistance. Because degradation in soil resistance due to cycling was thought to be primarily a phenomenon associated with an individual pile, Focht and Koch proposed that a reasonable solution for pile groups with cyclic loading could be obtained using p-y curves

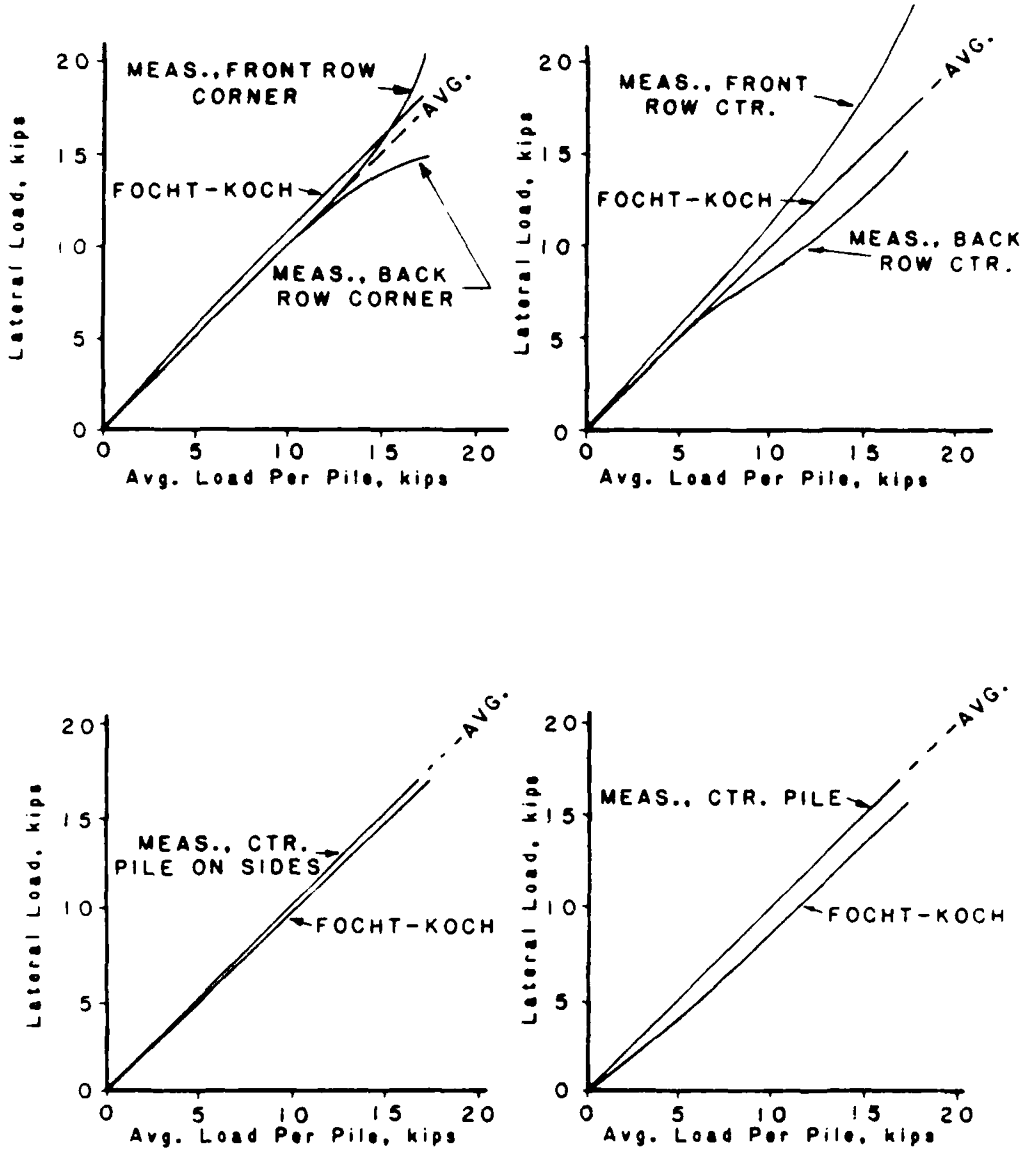


Fig. 8.9 Distribution of Load by Location, Cycle 1: Focht-Koch Prediction vs. Measured

computed for cyclic conditions and using y-multipliers to account for group effects as was done for the static case. The results of such an analysis for the conditions of the Houston test are discussed below.

Load-Deflection Predictions. The results of analyses using E_s = 2000 lb/sq in. are presented in Fig. 8.10. The trend is similar to that of the static loads and significant underprediction at large loads. The error in the analysis is seen to be larger for the cyclic case than for the static case. As discussed in Chapter 7, the effects of cyclic loading on the piles in the group were more significant than were the cyclic effects on the individual pile. The Focht-Koch method underpredicted deflections at large loads for the static case and, considering the fact that the method considers cyclic effects to be an exclusively local phenomenon, even larger errors in underprediction might have been anticipated for the cyclic case; such was indeed the case.

Load-Moment Predictions. Predictions of load vs. maximum moment for cyclic loading are presented in Fig. 8.11 and are again seen to follow a similar pattern to that of the static case. The range in which the moments predicted by the Focht-Koch method are conservative is reduced to average loads per pile of 5 kips or less, which is half of that for the static case. The use of a p-multiplier is again seen to have little effect. Plots of bending moment vs. depth in Fig. 8.12 indicate that the Focht-Koch procedure does not correctly model the increasing depth to maximum moment which occurs with increasing loads. An examination of p-y curves constructed using the Focht-Koch method will help to explain this deficiency.

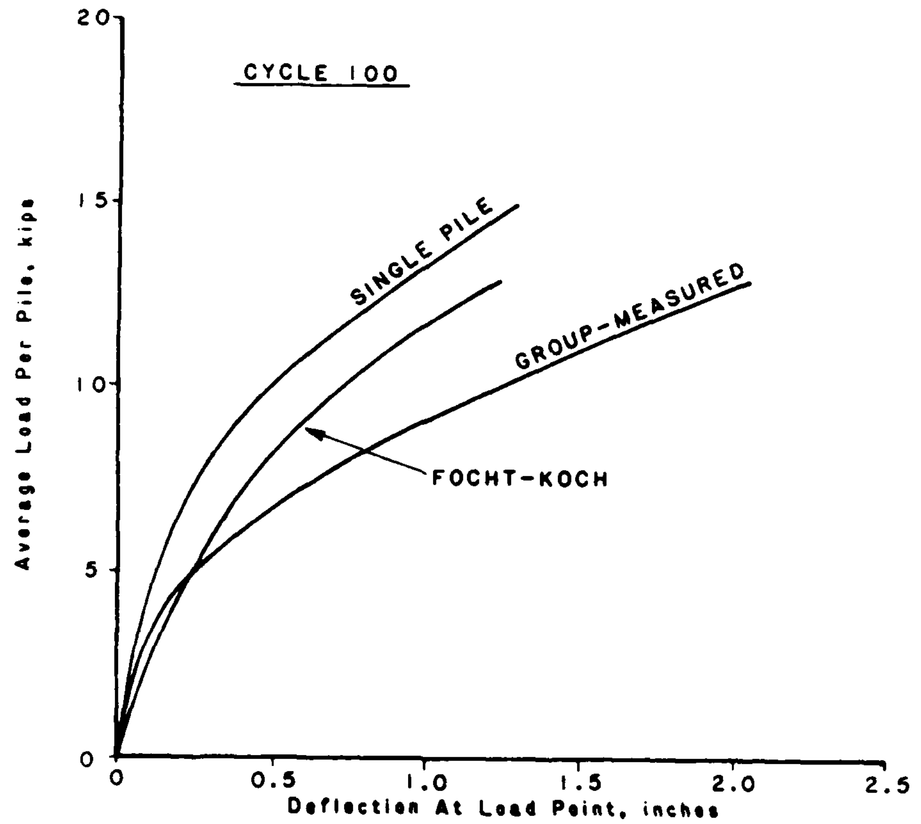


Fig. 8.10 Focht-Koch Lateral Load vs. Deflection Predictions, Cycle 100

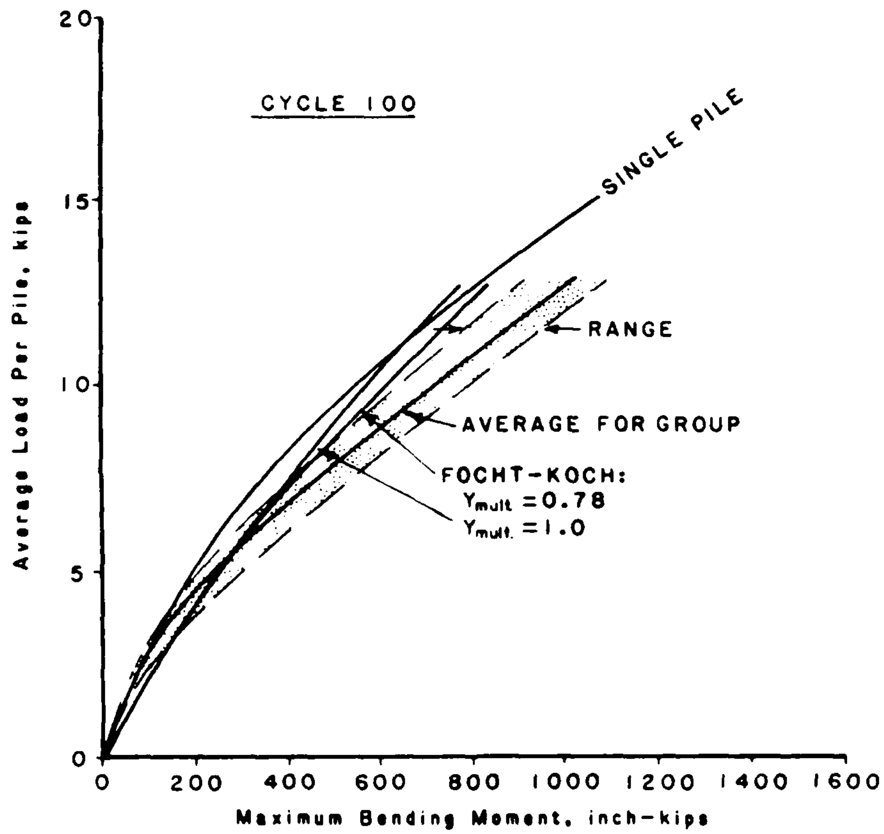


Fig. 8.11 Focht-Koch Load vs. Maximum Moment Predictions, Cycle 100

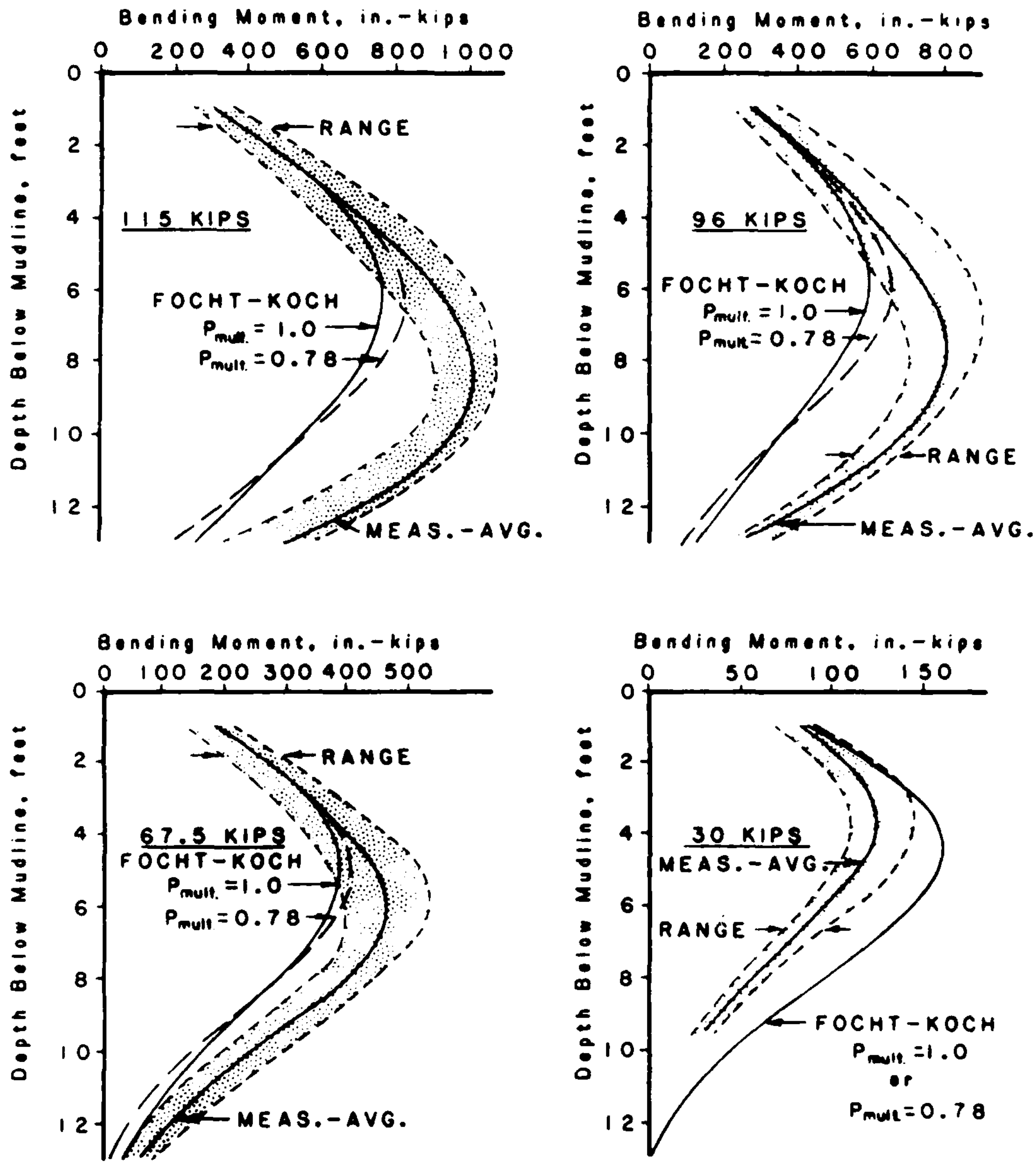


Fig. 8.12 Focht-Koch Bending Moment vs. Depth Prediction, Cycle 100

Predictions of p-y Curves. Presented in Fig. 8.13 are the predictions of p-y curves from the Focht-Koch method (using multipliers on the single pile curves from the on-site test) along with those derived from measurements for the 100th cycle at a given level of load. As with the static case, the Focht-Koch curves were constructed using y-multipliers applied to the single-pile curves. Different y-multipliers were used at each load that was analyzed in order to match the predicted deflections of the group presented earlier. A single y-multiplier is used to stretch all p-y curves along a pile at a given load.

The procedure of stretching the single p-y curve presents some problems for the case of cyclic loading. At fairly shallow depths, the deflection to reach the residual resistance of the soil for the single-pile p-y curve may be on the order of 1/2 in. or more. A y-multiplier of 8 would move that point to a deflection of 4 inches. It seems intuitively unreasonable that nearby piles moving about 1/2 in. would produce a 3-1/2 in. increase in deflection. For a curve at a relatively shallow depth, the effect of stretching may be actually to increase the soil resistance if equilibrium for the unstretched curve was achieved at a point beyond the peak. An examination of the Focht-Koch curves in Fig. 8.13 illustrates this situation. In order to match the predicted deflections of the group at large loads, the Focht-Koch p-y curves were stretched more and more at increasing loads, and the residual resistances of the soil were never attained. As for the static case, the maximum soil resistances derived from group pile measurements were substantially lower than those predicted,

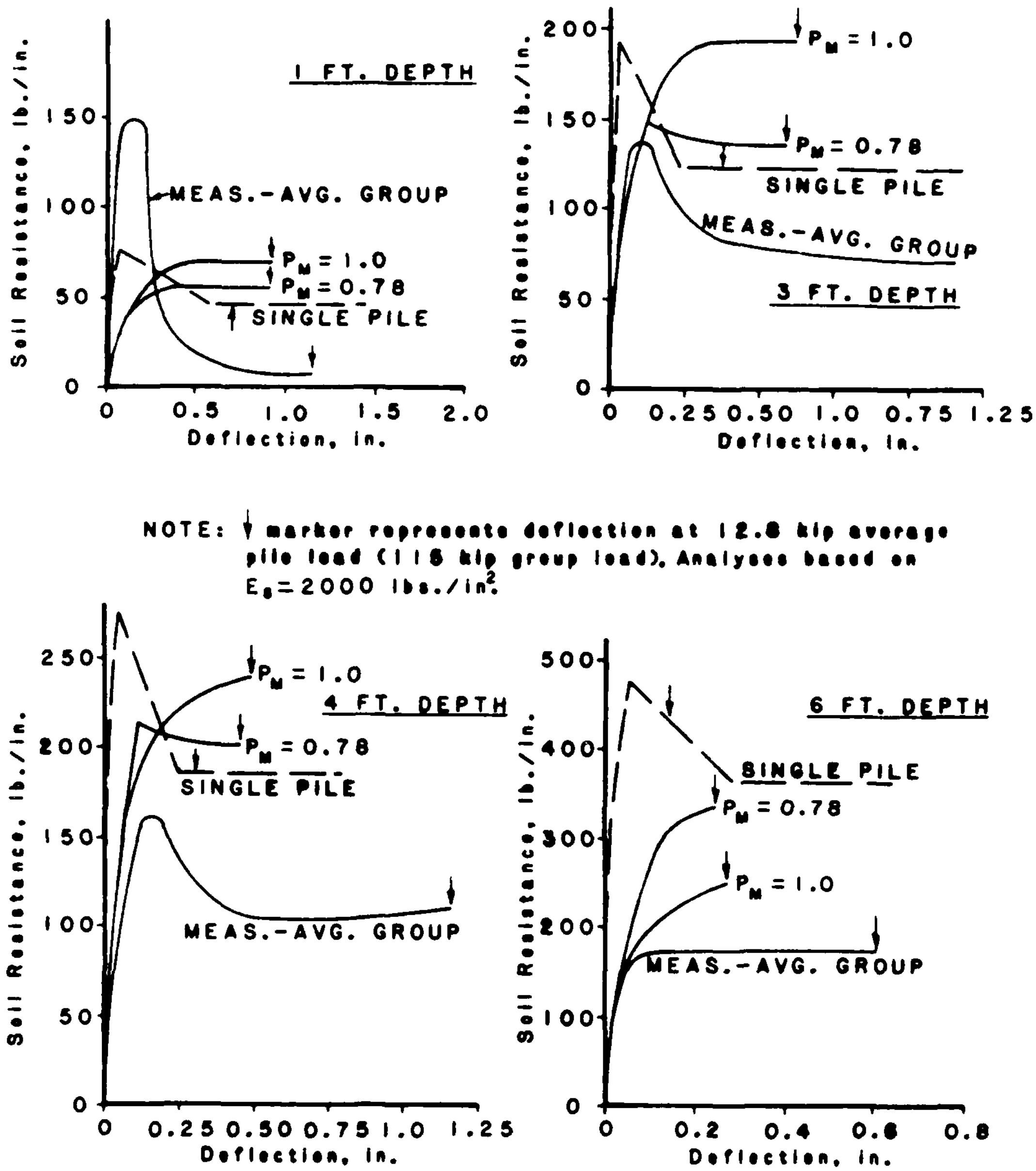


Fig. 8.13 Focht-Koch P-Y Curve Predictions, 100 Cycles

and the maximum soil resistances for the single pile were lower as well.

Distribution of Load in the Group. Distribution of load to the piles for the static case was shown to indicate that all of the loads were fairly close to the average value. The stretching factors used in the Focht-Koch analysis at a given load are higher for cyclic conditions than for static (because the deflection predicted for a single pile under cyclic conditions is higher). The result of these higher factors is to distribute the predicted loads for cyclic conditions even more uniformly.

Summary

The Focht-Koch procedure is the earliest of the hybrid models for estimating behavior of a pile group under lateral loads and provides the advantage over the elastic model of accounting for the effects of nonlinear resistance of the soil in the region near the individual piles. The method also attempts to predict in a rational manner the distribution of stresses with depth. Comparisons with measured results indicate that the method is unconservative in predicting deflections, maximum bending moments, and depth to maximum bending moments at loads in excess of working loads for both static and cyclic loadings. The method does not account for the observed large reductions in maximum soil resistances in the piles in the group with respect to single-pile values and does not properly model the loss of soil resistance due to cyclic loading.

PILGP2R

The unreasonable offsets in p-y curves for the shallow soils produced by the use of the constant y-multiplier with depth in the Focht-Koch method prompted more analytical studies using hybrid models. PILGP2R represents possibly the most complete hybrid model using load-transfer curves and elastic interaction between piles. As described in Chapter 4, the program computes displacement at the location of each individual p-y curve due to point loads at each pile increment throughout the group. These displacements are used to derive a y-multiplier for each individual p-y curve in the group. The approach of PILGP2R follows the basic idea proposed by Focht and Koch in that nonlinear soil response is thought to be due to plastic strains in the near field around each individual pile, and group effects are thought to be due to superimposed displacements caused by the nearby piles. Because strains in the far field around the piles are thought to be small, the superimposed displacements are computed on the assumption of an elastic soil using a modulus representative of small strains.

The maximum modulus corresponding to small strains can be determined by using in-situ techniques such as measurement of velocity of shear waves using the cross-hole method. Such measurements made at the test site indicated a Young's modulus of about 30,000 lb/sq in. that was relatively constant with depth. Other recommendations for values of modulii to be used in hybrid models in clay have been summarized by O'Neill (1983) and generally range from 300 to 1000 times the value of undrained shear strength as determined from UU tests. Values

of $400S_u$ and $800S_u$ were used in the analysis that follows; some smaller values were also used to examine the influence of the soil modulus. All p-y curves used in PILGP2R were those derived from the single-pile test on the specific site.

Static Case

Load-Deflection Predictions. Presented in Fig. 8.14 are the results of analyses using the aforementioned values of E_s, as well as results using two lesser values. As for the Focht-Koch method, the analysis underpredicts deflections by a substantial amount at loads in excess of typical working loads. The results are somewhat better than the Focht-Koch predictions for the $E_s = 800S_u$ case, particularly in the range of small loads (less than about 7 kips). The results were not particularly sensitive to E_s for reasonable values of $400S_u$ or more. The curves shown in Fig. 8.14 demonstrate that the observed behavior of the group cannot be matched throughout the entire range of load by simply adjusting the values of E_s that are used in the analysis.

Load-Moment Predictions. Curves giving load vs. maximum moment for the PILGP2R analysis are presented in Fig. 8.15. The values that are shown are for the pile with the largest load as predicted by the method, although all of the loads in the piles were predicted to be within 2% of the maximum for all but the very small loads. These data reveal that maximum moments that are predicted for the group are very close to those of the single pile for any reasonable value of E_s, and thus are underpredicted by a significant amount for the group at large loads. However, the measured values for the group

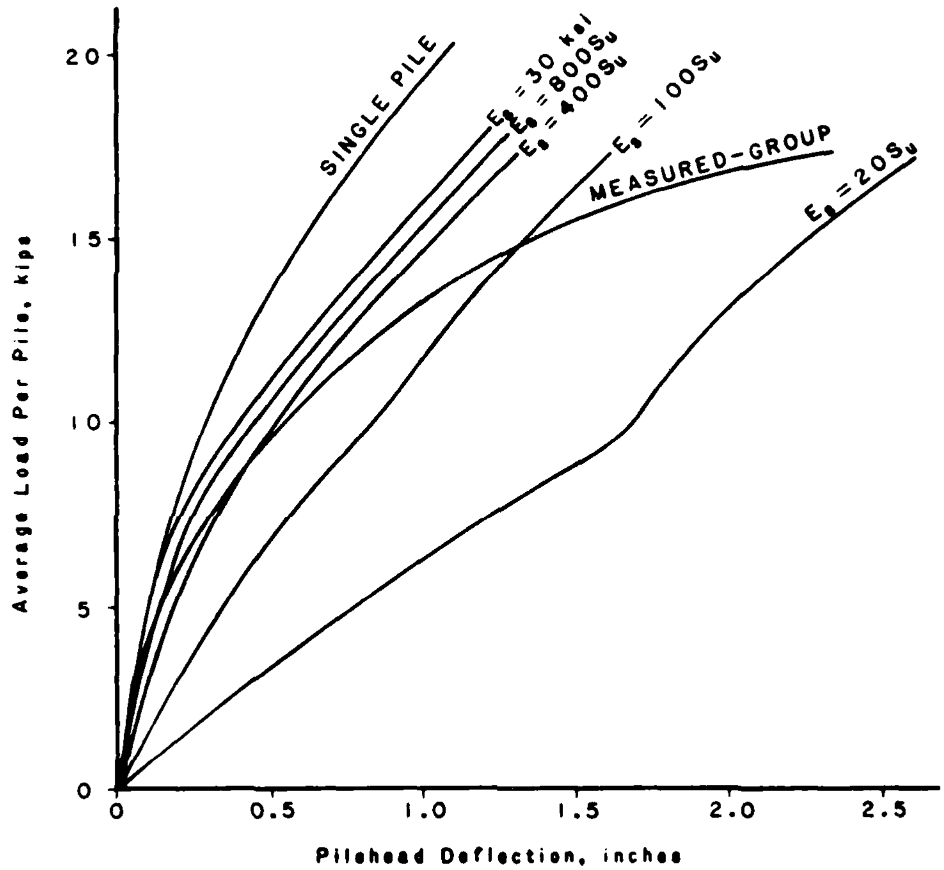


Fig. 8.14 PILGP2R Lateral Load-Deflection Predictions, Cycle 1 (Static)

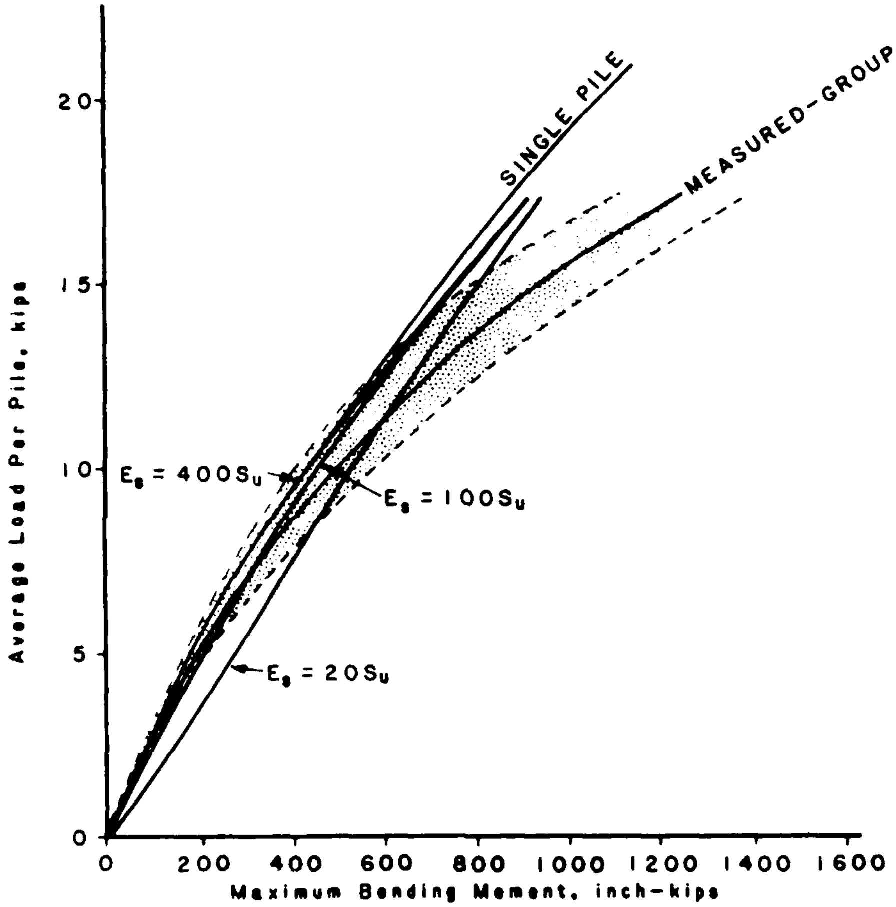


Fig. 8.15 PILGP2R Load vs. Maximum Moment Predictions, Cycle 1 (Static)

were also similar to the single-pile values for loads less than about 7 or 8 kips per pile. The PILGP2R predictions in the range of 7 to 8 kips are thus much better than those of the Focht-Koch method, which overpredicted moments at small loads.

The distribution of moments with depth for several loads as measured and as predicted by PILGP2R are illustrated in Fig. 8.16. A value of E_s of $400S_u$ was used for these and subsequent comparisons. Other values for E_s are shown on the plot for 156 kips, and it is evident that the predicted moments (particularly the maximum moments and the depth to maximum moment) are not greatly sensitive to E_s for any reasonable values. The pattern of predicted moments is similar to the Focht-Koch predictions in that the value of and depth to maximum moment are underpredicted for larger loads. The predicted depth to maximum moment varies little with increasing load, in contrast to the measured depth to maximum moment that progressed much deeper at larger loads.

<u>Predictions of p-y Curves</u>. Presented in Fig. 8.17 are the predicted p-y curves for several depths, along with those derived from measurements from the group and from the single pile. The predicted curves are based on $E_s = 400S_u$. PILGP2R predicts p-y curves very similar to those of a single pile, but with some offset in deflection. As was the case with the Focht-Koch method, the major source of error in the predicted curves, as compared with measurements, is the inability of the method to account for a reduction in the maximum soil resistance due to group effects.

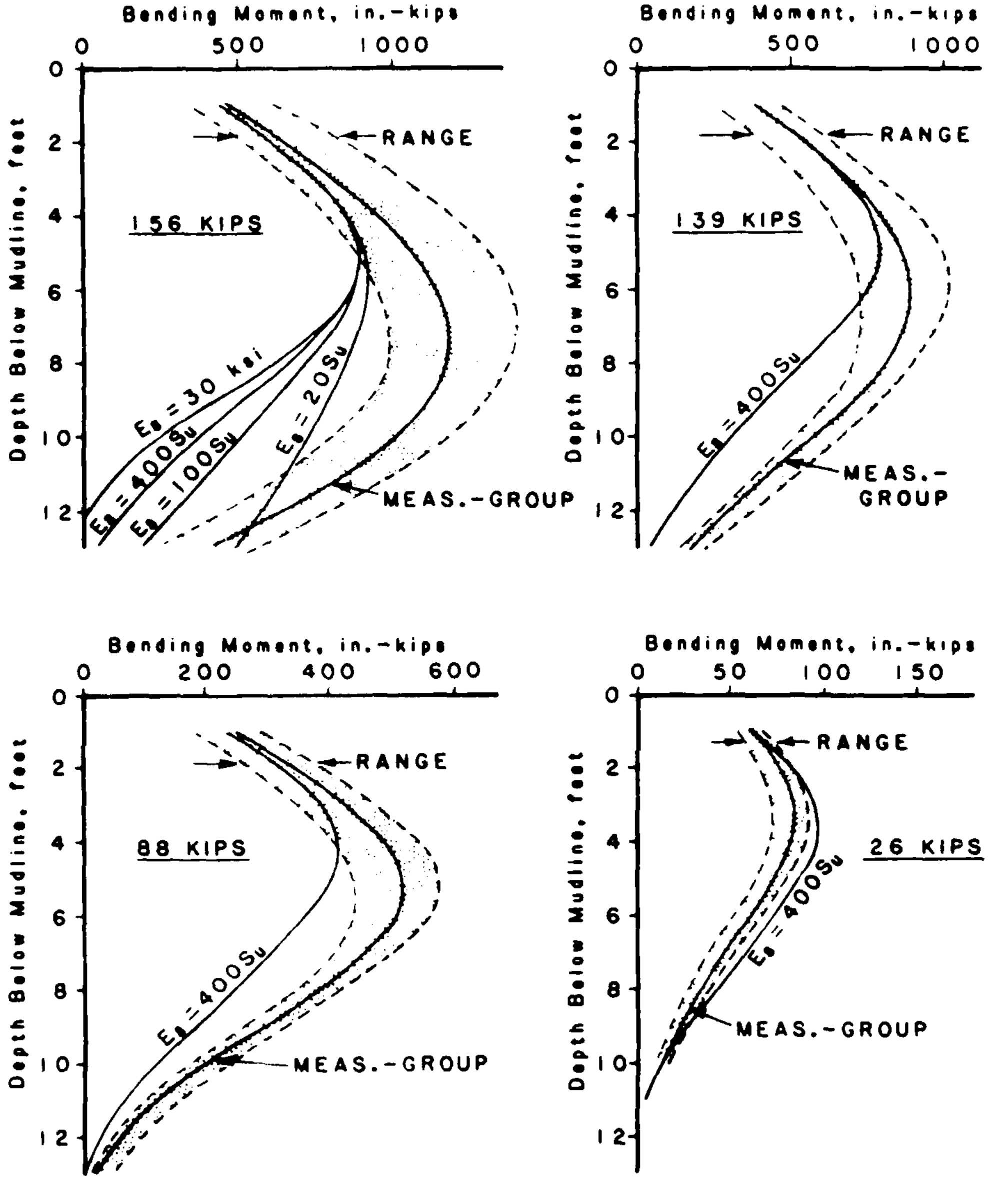
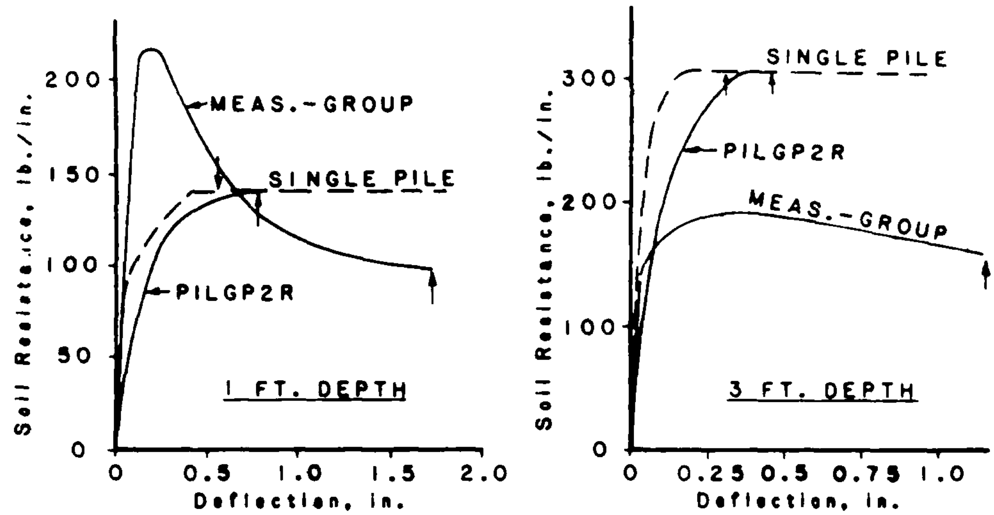


Fig. 8.16 PILGP2R Bending Moment vs. Depth Predictions, Cycle 1 (Static)



NOTE: ↑ marker represents deflection at 17.3 kip average pile load (156 kip group load)

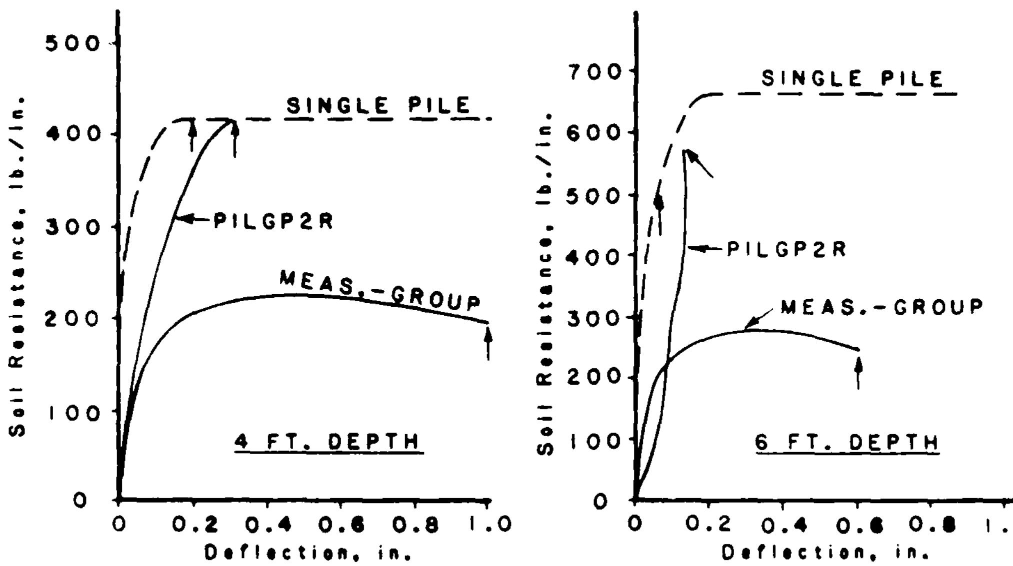


Fig. 8.17 PILGP2R P-Y Curve Predictions, Cycle 1 (Static)

Distribution of Load in the Group. Loads to the individual piles in the group were predicted by PILGP2R to be within about 0.3 kips of the average for all piles and for all loads that were analyzed. These loads are thus more evenly distributed than was predicted by Focht-Koch. Measurements indicated that the front-row piles carried a greater portion of the load in the field test.

Cyclic Case

As with the Focht-Koch procedure, PILGP2R makes use of strain-softening p-y curves to model degradation during cyclic loading and modifies these curves for group effects in a fashion similar to that used for static loads. The constant y-multiplier at all depths and the resulting large offsets in deflection for p-y curves near the mudline caused problems in modelling cyclic degradation with the Focht-Koch procedure; the residual values of soil resistance were never achieved for shallow depths. PILGP2R does not suffer from this constant y-multiplier restriction and perhaps could be expected to model better cyclic degradation in the p-y curves. Results of analyses for the cyclic loadings used in the Houston test follow. All analyses are based on $E_s = 400S_u$.

Load-Deflection Predictions. The load-deflection curves for the 100th cycle for the group and for the single pile are presented in Fig. 8.18, along with predictions from PILGP2R. There was a certain range of loads in which PILGP2R would not converge to a solution. At loads slightly lower than this range, deflections seemed to be anomalously low and were even lower than the single-pile deflections for some loads. Examination of the mode curves for the individual piles

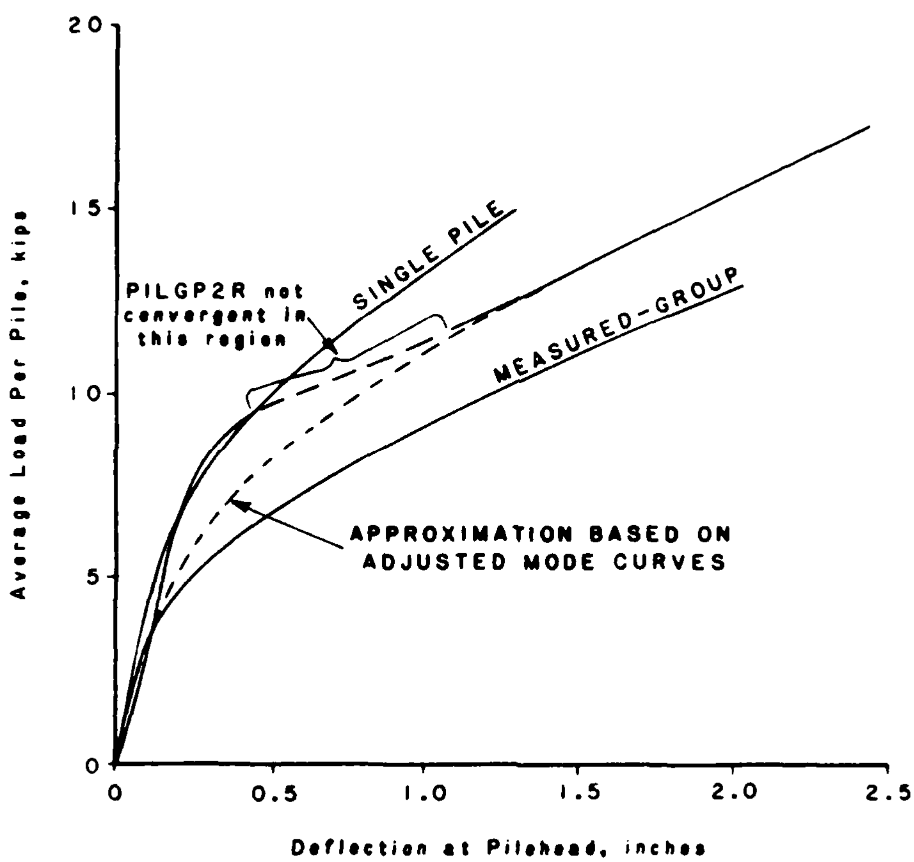


Fig. 8.18 PILGP2R Lateral Load vs. Deflection Predictions, Cycle 100

(Chapter 4 presents an explanation of mode curves and the method of solution in PILGP2R) indicated that the assembly of the stiffnesses of the individual pileheads and solution of the group-matrix problem resulted in a deflection for the piles in the group less than what was compatible with the mode-curve deflections. Once adjusted for group effects, the mode curves are reportedly not changed during the assembly-and-solution process, so the incompatibility is presently unexplained. Because the loads to the individual piles were very close to the average load in every case, a deflection could be picked off the mode curves for the individual piles for the average load and plotted to produce the dotted line in Fig. 8.18. The dotted line appears to be a more reasonable prediction of behavior in the region indicated. This behavior leads to the suspicion that there may be an error in the PILGP2R algorithm under certain conditions probably involving the fit of cubic spline functions to the mode curve data points. As will be subsequently shown, the region of computational instability occurs at the point at which many of the near-surface p-y curves are in the transition between the peak and the residual soil resistance (i.e., the range of deflections for which the p-y curve has a negative slope). Nevertheless, the algorithm seemed to work correctly for small loads and for the maximum loads achieved during the load test. PILGP2R predicted deflections very well at small loads, and underpredicted deflection at large loads by a substantial amount. It is of interest to note again that the analysis was performed using p-y curves derived from a single-pile test that was very near the group,

so that very good agreement should be realized if the analysis is correctly accounting for group effects.

Load-Moment Predictions. Presented in Fig. 8.19 are the predictions of load vs. maximum moment of PILGP2R. As discussed previously, the solution for loads from about 5 to 11 kips per pile was either nonconvergent, or otherwise suspect. At larger and smaller loads, PILGP2R predicts maximum moments very close to those of a single pile. Figure 8.20 provides plots of bending moment vs. depth for several loads. The predicted distribution for the 67.5-kip load is suspect, but those for 30 and 115 kips are thought to be correct. As was the case for static conditions, PILGP2R underpredicts for large loads both the maximum moment and the depth to maximum moment. The solution appears to be fairly close to measurements for the 30-kip load.

Predictions of p-y Curves. Presented in Fig. 8.21 are the p-y curves that were predicted by PILGP2R along with those derived from measurements for the 100th cycle at a given level of load. The range of loads in which the solution was unstable coincided with the transition zone from peak to residual loads for the shallow p-y curves. The stable solution for the larger loads was achieved at a soil resistance equal to residual values, as evident by the marker for the 115-kip load. PILGP2R tracked the single-pile p-y curves fairly closely for shallow curves, with an increasing offset in deflection at the 6-ft depth. In contrast to the Focht-Koch procedure, PILGP2R modelled the degradation of soil resistance to the residual values for the single pile. However, the significant reduction in both the peak value and residual value of soil resistance was not predicted.

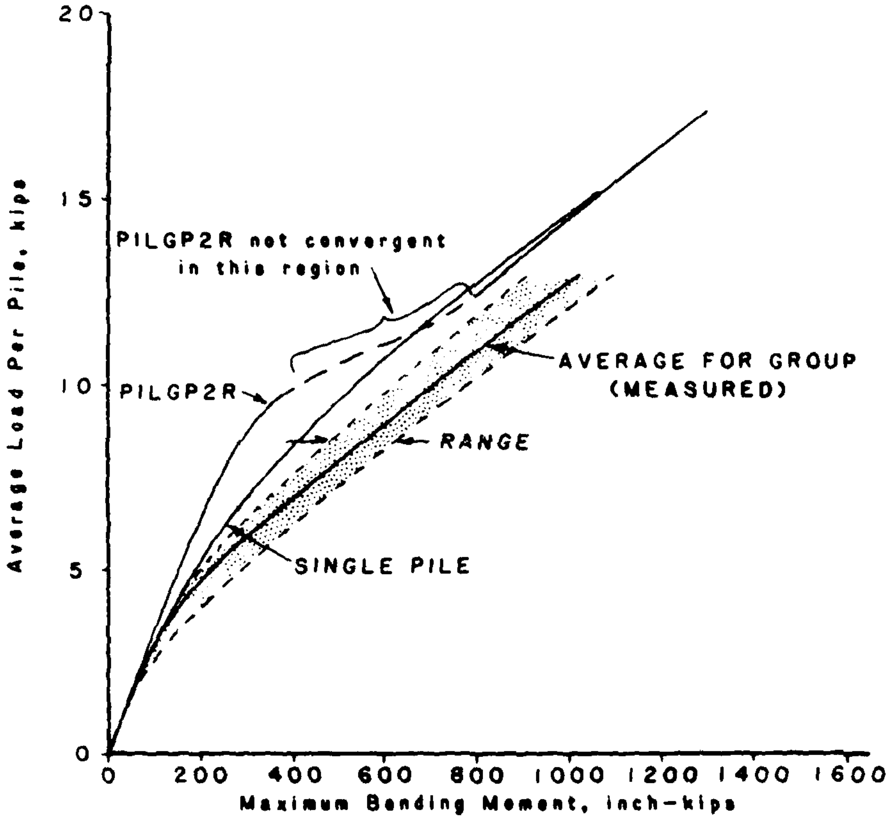


Fig. 8.19 PILGP2R Lateral Load vs. Maximum Moment Predictions, Cycle 100

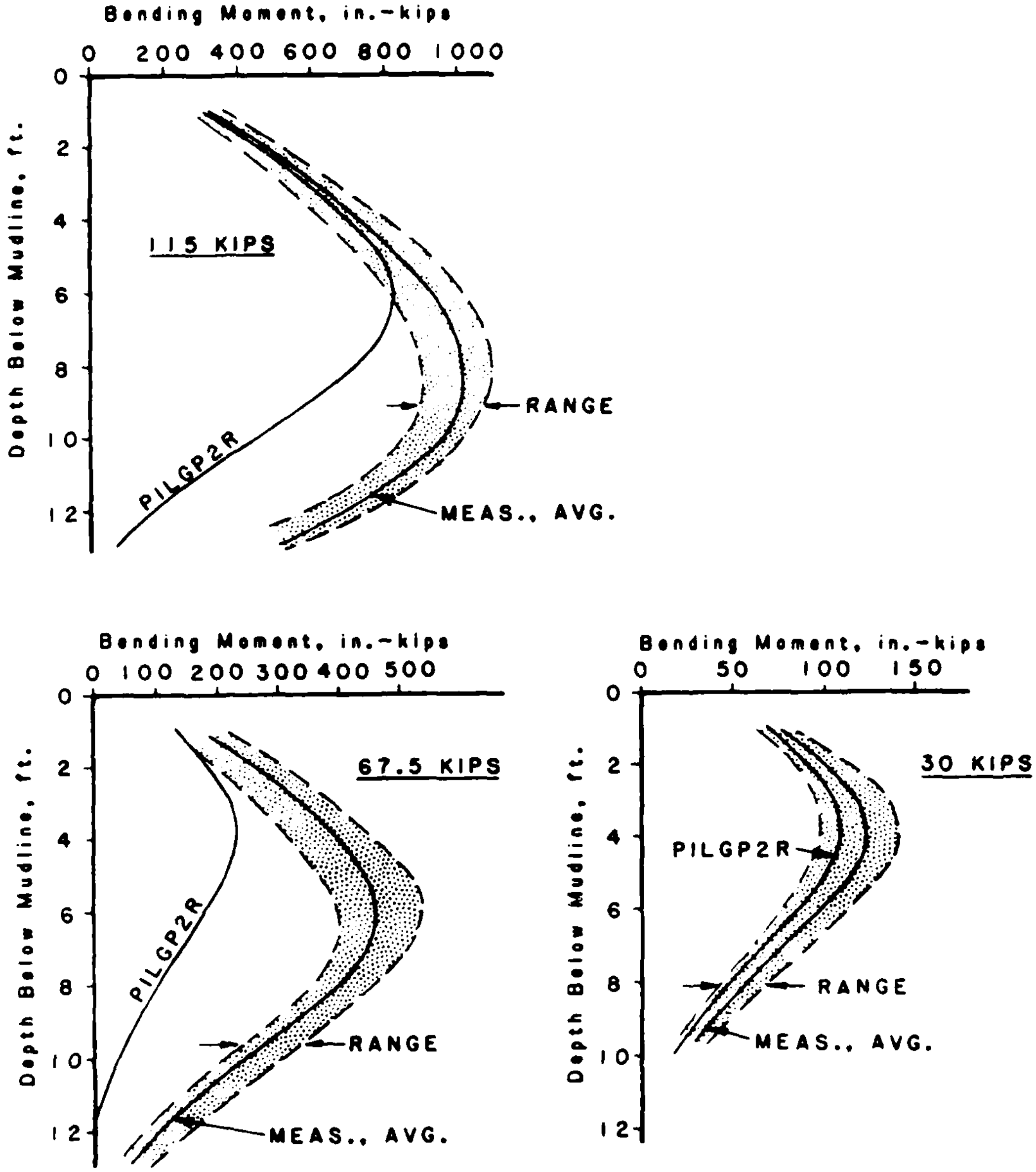


Fig. 8.20 PILGP2R Bending Moment vs. Depth Prediction, Cycle 100

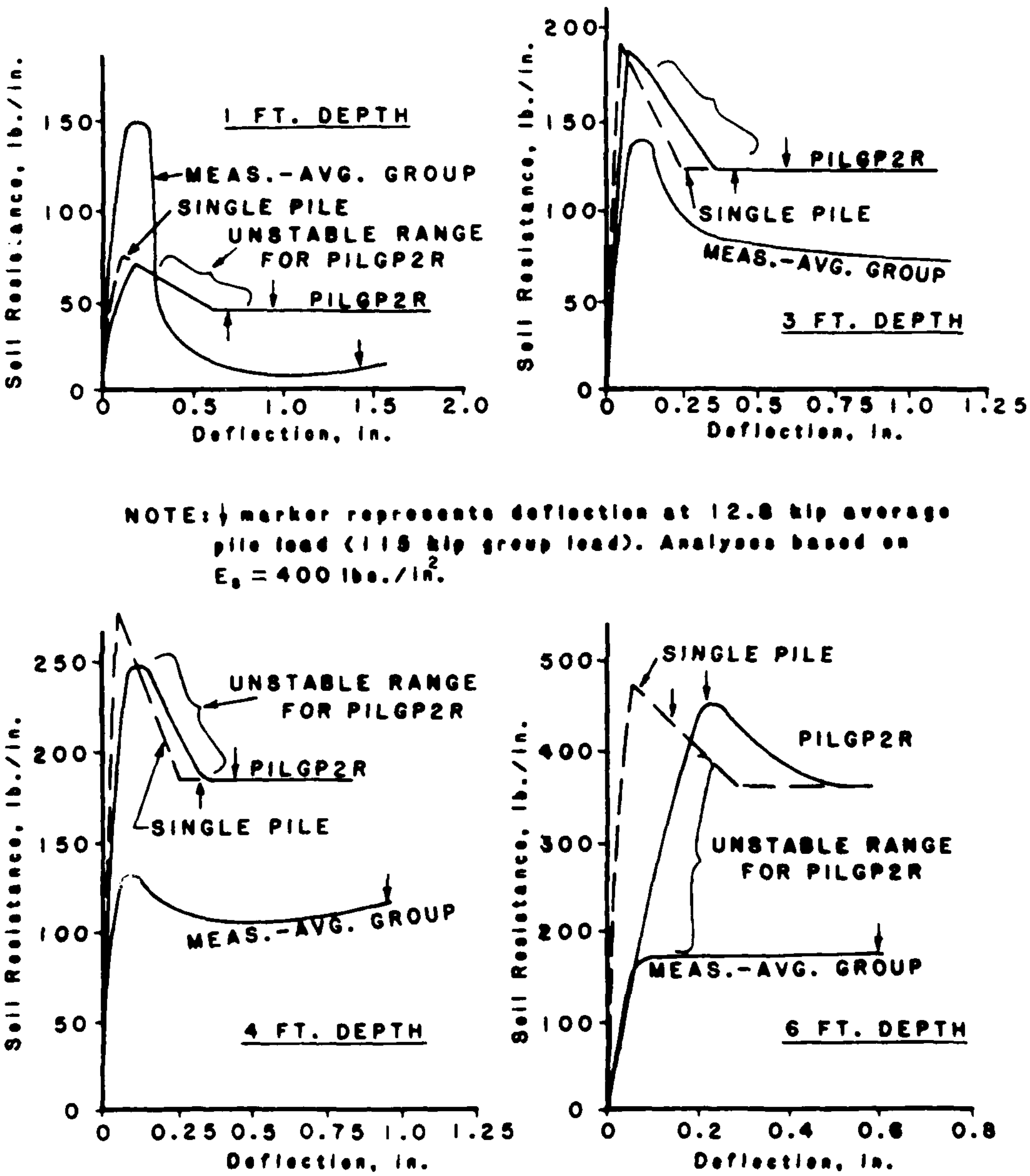


Fig. 8.21 PILGP2R P-Y Curve Predictions, 100 Cycles

Distribution of Load in the Group. As for the static case, PILGP2R predicted that all piles in the group would be loaded to within 0.3 kips of the computed average.

Summary

PILGP2R was designed to be an improvement over the Focht-Koch-hybrid model in that it more appropriately accounted for pile-soil-pile interaction through an elastic soil. As the results of analyses show, PILGP2R did in fact model the test results better than Focht-Koch in several respects. PILGP2R predicted load-deflection, load moment, and distribution of moment with depth much more closely than Focht-Koch for loads less than about 4 kips per pile. PILGP2R did not accurately model the response of the group in a general sense. Deflections, bending moments, and depth to maximum moment were seriously underpredicted at large loads for both static and cyclic conditions. These inadequacies can be attributed to a failure to account for reductions in the maximum soil resistance due to group effects. The method also did not properly predict the distribution of load to the piles. The fact that test results indicated a substantial reduction in maximum soil resistance as well as a load bias toward the front-row piles must cast serious doubt on the validity of using an elastic soil to model pile-soil-pile interaction at lateral loads in excess of very small values.

SINGLE-PILE METHOD

The single pile method described in Chapter 4 is not really a formal method of analysis which has been described and reviewed in the

technical literature, but rather a means of providing an upper bound on the predicted response of a pile group. All of the piles in the group, along with the soil within the group are assumed to act as one unit. This assumption should thus represent the extreme of group interaction in which the behavior of an individual pile is insignificant. The response is computed by analyzing a large, imaginary pile which has a flexural stiffness equal to the sum of the stiffnesses of the individual piles. The diameter of the imaginary pile is taken as the diameter of a circular pile having the same perimeter as that of the group. For this case, that diameter is 85 inches.

Curves of p vs. y for the imaginary pile have been computed using two different procedures. The first is the procedure recommended by Matlock (Matlock, 1970) and described in API RP2A for use in soft clay. Neither the Matlock recommendations nor any other available procedure predicted the response of the single pile with any degree of accuracy. Matlock's procedure for p-y construction (referred to as SO in this report) has been used primarily as a basis for comparison with the second procedure that is used, called the "site-specific" procedure (referred to as SS in this report). The SS p-y curves were generated by modifying the constants in the Matlock equations to produce agreement with the results of the test of the single pile. The approach taken has the advantage of matching the response of the small pile. The p-y curves computed by both methods for the large imaginary pile also provided the necessary first step in the Bogard-Matlock procedure, that is described later. Results of analyses for the imaginary pile follow.

Static Case

Load-Deflection Predictions. Presented in Fig. 8.22 are the results of computations for an 85-in. diameter pile using the two methods of constructing p-y curves described earlier. The loads used in computations for the large, imaginary pile have been divided by 9 to plot the average load per pile. Both methods greatly overpredict deflection for a given load. As will be evident in the later discussion of the Bogard-Matlock method, the stiffer response computed using the SO criteria is attributable largely to the larger values of maximum soil resistance at shallow depths predicted by this criterion.

Load-Moment Predictions. The predictions of maximum moment computed by the single-pile method using the two criteria are presented in Fig. 8.23. Values of moment that are shown are computed values for the imaginary pile divided by the number of piles in the group. Compared to the measured value, the predictions of maximum moment are quite good; the prediction using the SS criterion matches the shape of the upper bound of the measured response very well. These plots indicate that the prediction of maximum bending moments is less sensitive to the computation procedures used than are deflections.

Cyclic Case

The degradation in soil resistance is thought to be due primarily to the effects of gapping and scour immediately around a pile; it does not appear rational to consider gapping and scour around an 85-in. diameter imaginary pile to be relevant for a group of piles where the spacing is 3 diameters on center.

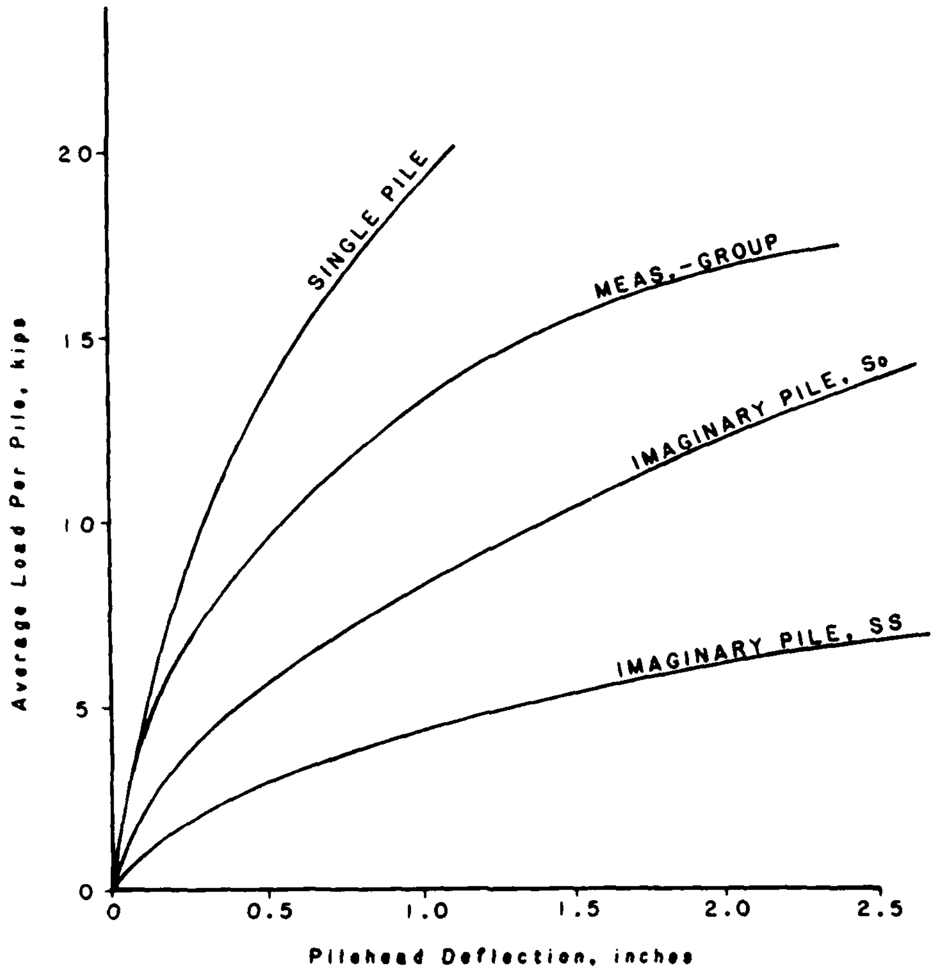


Fig. 8.22 Single Pile Method Load-Deflection Predictions, Cycle 1 (Static)

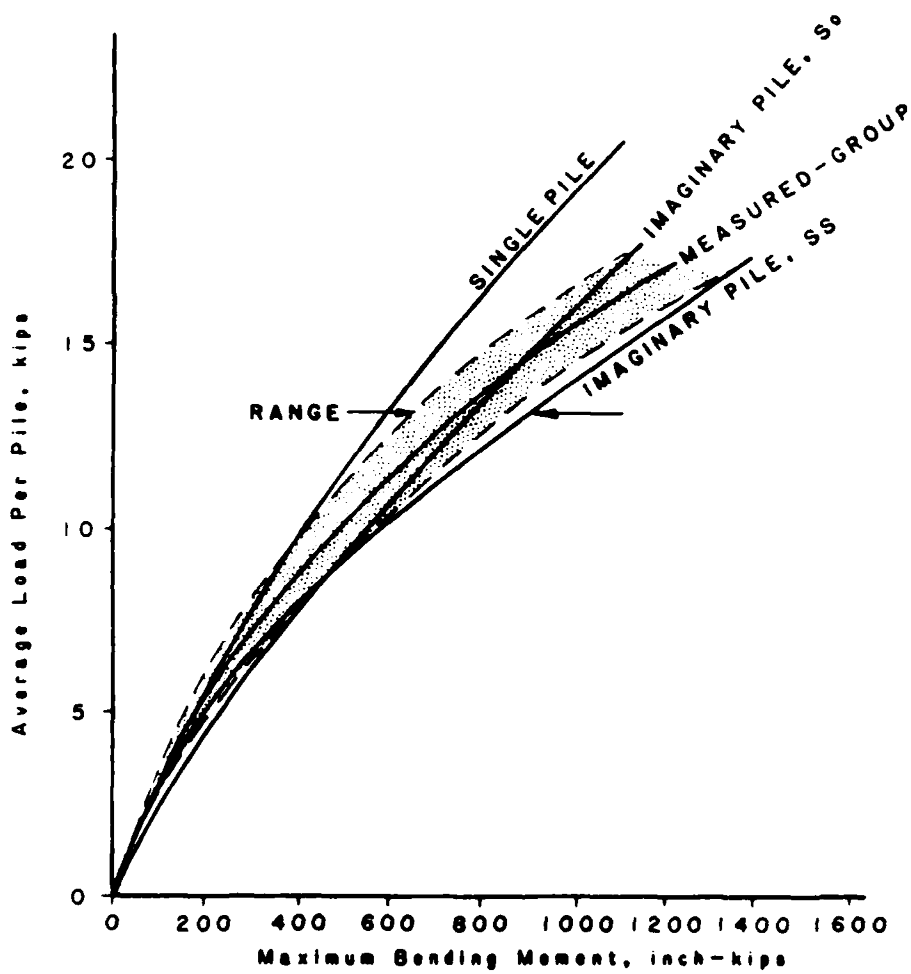


Fig. 8.23 Single Pile Method Load vs. Maximum Moment Predictions, Cycle 1 (Static)

Summary

The single-pile method of treating a pile group should offer an upper limit on static deflection and moments if the response of a single pile can be predicted by the p-y curves that are employed. The effect of diameter on p-y curves is a parameter that obviously influences the predictions with the single-pile method. The fact that the p-y procedure that is used is largely empirical, coupled with the general lack of experimental data for piles of varying diameter in similar soil, does not contribute to confidence in the results of such an approach. The single-pile method seemed to provide an upper-bound solution for the results of this study, as the deflections were conservatively predicted. The single-pile method using the site-specific p-y curves predicted maximum bending moments conservatively.

BOGARD-MATLOCK METHOD

Errors in the predictions made with the hybrid models were noted to be largely due to the fact that the maximum values of soil resistance (p_{ult}) predicted for the piles in the group were too high. The hybrid models did not provide any rational basis for reducing p_{ult} from the single-pile values to account for group effects. Bogard and Matlock apparently made similar observations in evaluating the results of the Harvey, Louisiana, test and proposed a method to accommodate the reduction in p_{ult}. The basic procedure that is used in the Bogard-Matlock method is outlined in Chapter 4. Simply stated, the method combines the single p-y curves with a modification of the p-y curves for a large, imaginary pile (similar to that described in the

previous section) to produce p-y curves for the analysis of piles in the group. All piles in the group are assumed to behave in the same manner. Although the authors recognize that such behavior is not necessarily the case (their procedure is not represented as a rigorously correct solution), they propose that variations are small and can adequately be accommodated by a small overdesign.

Because the Bogard-Matlock method involves the superposition of two p-y curves to produce another, the results of the analysis are quite sensitive to the methods used for generating these p-y curves. As indicated in the discussion of the single-pile method, none of the available methods of predicting p-y curves yielded results for the test of the single pile with an acceptable level of accuracy. The Bogard-Matlock procedure has thus been performed using both the API RP2A recommendations for p-y curves after Matlock (1970) and a modification of that p-y criterion that was made in order to match the results of the test of the single pile. These two criteria are labelled SO and SS, respectively, as noted earlier. The modified p-y curves (SS) are thought to represent best the use of the method in principle because the use of those curves actually will yield results that match results of the test of the single pile. Thus, any errors in the predictions of the behavior of the group should be solely due to the analytical method. However, the SS p-y curves, derived from a modification of the SO curves, are not unique in that other sets of p-y curves could also be used to match the single-pile data. The use of a different form for the p-y curves would result in different predictions for the imaginary pile and subsequently would produce differ-

ent predictions for the group. Such is always the problem with a method so dependent on empirical correlations; the approach used herein represents an attempt to follow Bogard and Matlock's concepts as closely as possible and still provide feedback on the validity of the approach.

For the spacing of the piles in the Houston test group, the p-y curves from the imaginary pile dominated the group-pile predictions. The resulting reduction in p_{ult} from the single-pile-predicted values was found to be substantial for each of the two methods. Figure 8.24 presents a plot of these p_{ult} values as predicted and as measured for both the group and the single pile under static conditions. Because of the overprediction of p_{ult} by the SO method for shallow depths for the single pile, it is not surprising that p_{ult} was overpredicted for the group. The SS method underpredicted p_{ult}, but the trend of p_{ult} with depth was very similar to the measured values.

Although the predictions of p_{ult} are a major factor controlling the prediction of response of the group, the predicted shape of the p-y curves is also an important factor; the shape used in the SS method was not the same as that of the SO method. The important problem of cyclic response is considered in the Bogard-Matlock method and has been analyzed using site-specific (SS) p-y curves. Results of the analyses and further discussion of important parameters influencing the results are presented in the paragraphs that follow.

Static Case

Load-Deflection Predictions. The load-deflection predictions for the group for each method are presented in Fig. 8.25, along with

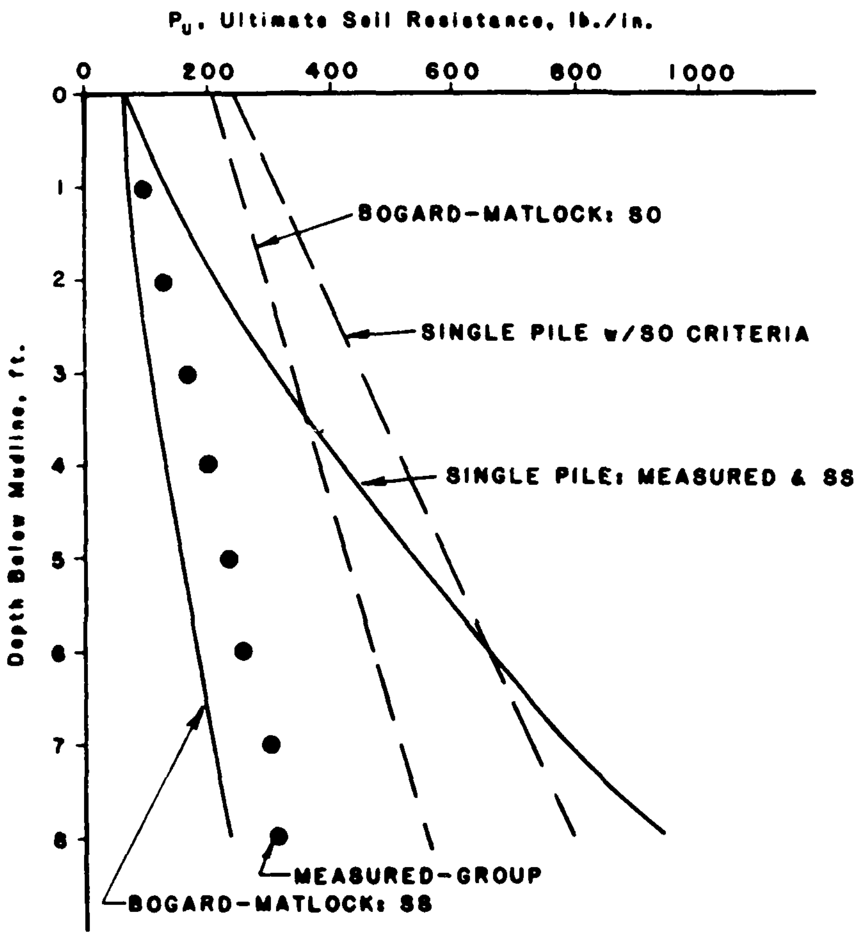


Fig. 8.24 Bogard-Matlock Predictions of Ultimate Soil Resistance vs. Depth, Cycle 1 (Static)

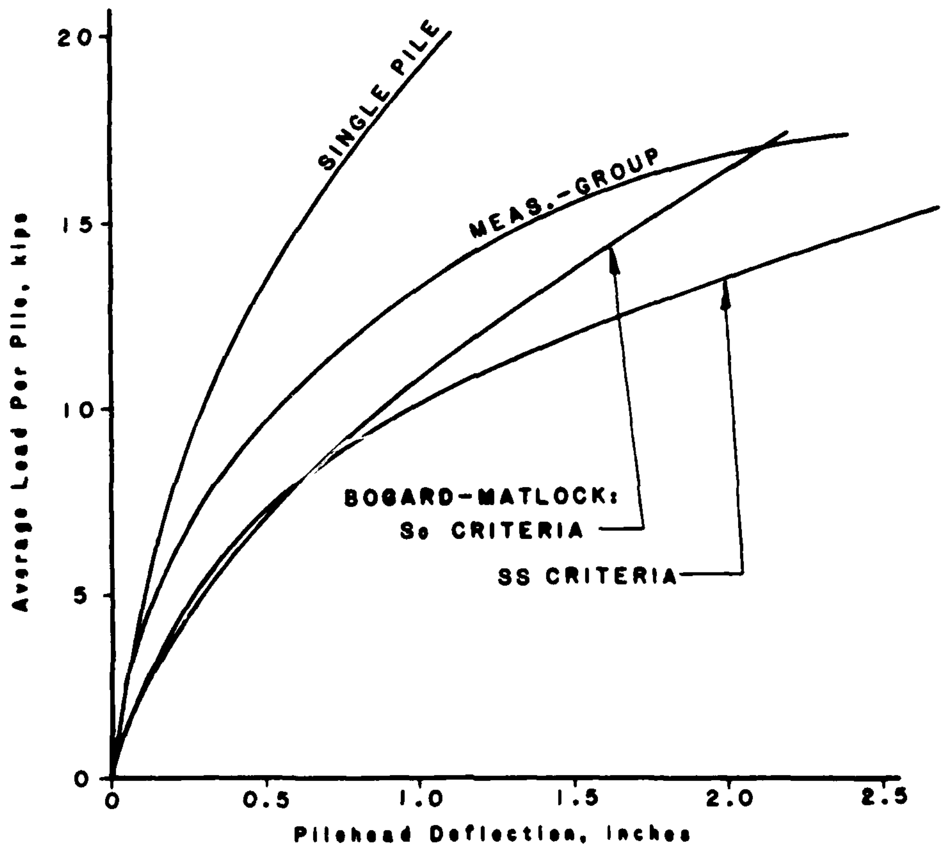


Fig. 8.25 Bogard-Matlock Lateral Load-Deflection Predictions, Cycle 1 (Static)

measured values. The SS predictions are seen to match the shape of the curve fairly well, but the deflections are overpredicted throughout the range of loading. The general similarity of the shape of the curves implies that the nonlinear interaction of the group is modelled to some extent.

Load-Moment Predictions. The predictions of load vs. maximum moment for the Bogard-Matlock model are shown in Fig. 8.26. The SS method is seen to model very closely the maximum moments that were measured in the Houston test at all values of load. Presented in Fig. 8.27 are the moment vs. depth distributions at several selected loads. This figure indicates that although the maximum moments are close to the measured values, the depth to maximum moment is overpredicted by about 30%. Unlike the hybrid models, this method correctly predicts the significant deepening of the maximum moments with increasing load.

Predictions of p-y Curves. Presented in Fig. 8.28 are the p-y curves for the Bogard-Matlock method, along with values derived from measurements. The points made earlier regarding the underprediction of p_{ult} with the Bogard-Matlcok procedure using the SS method are evident in these plots. The shape of the predicted p-y curves appears to be fairly close to those of the measured curves, except that the p_{ult} values are too low.

Distribution of Load in the Group. As mentioned previously, the Bogard-Matlock approach assumes the load to be evenly distributed among all the piles in the group.

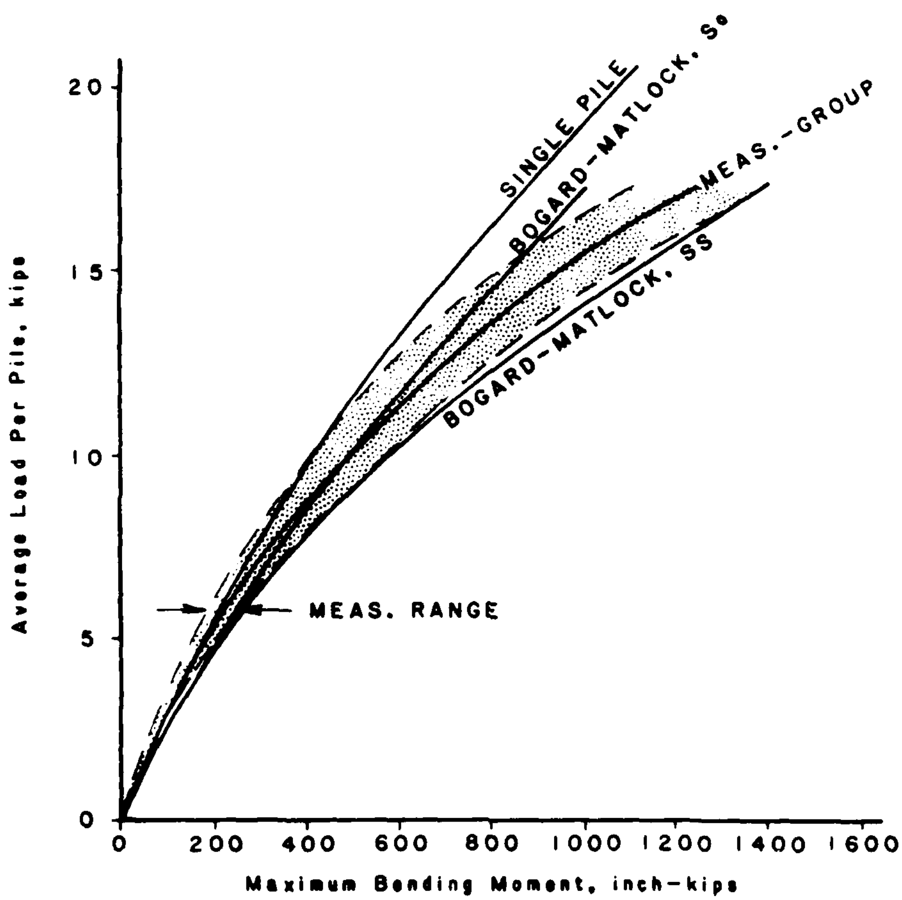


Fig. 8.26 Bogard-Matlock Load vs. Maximum Moment Predictions, Cycle 1 (Static)

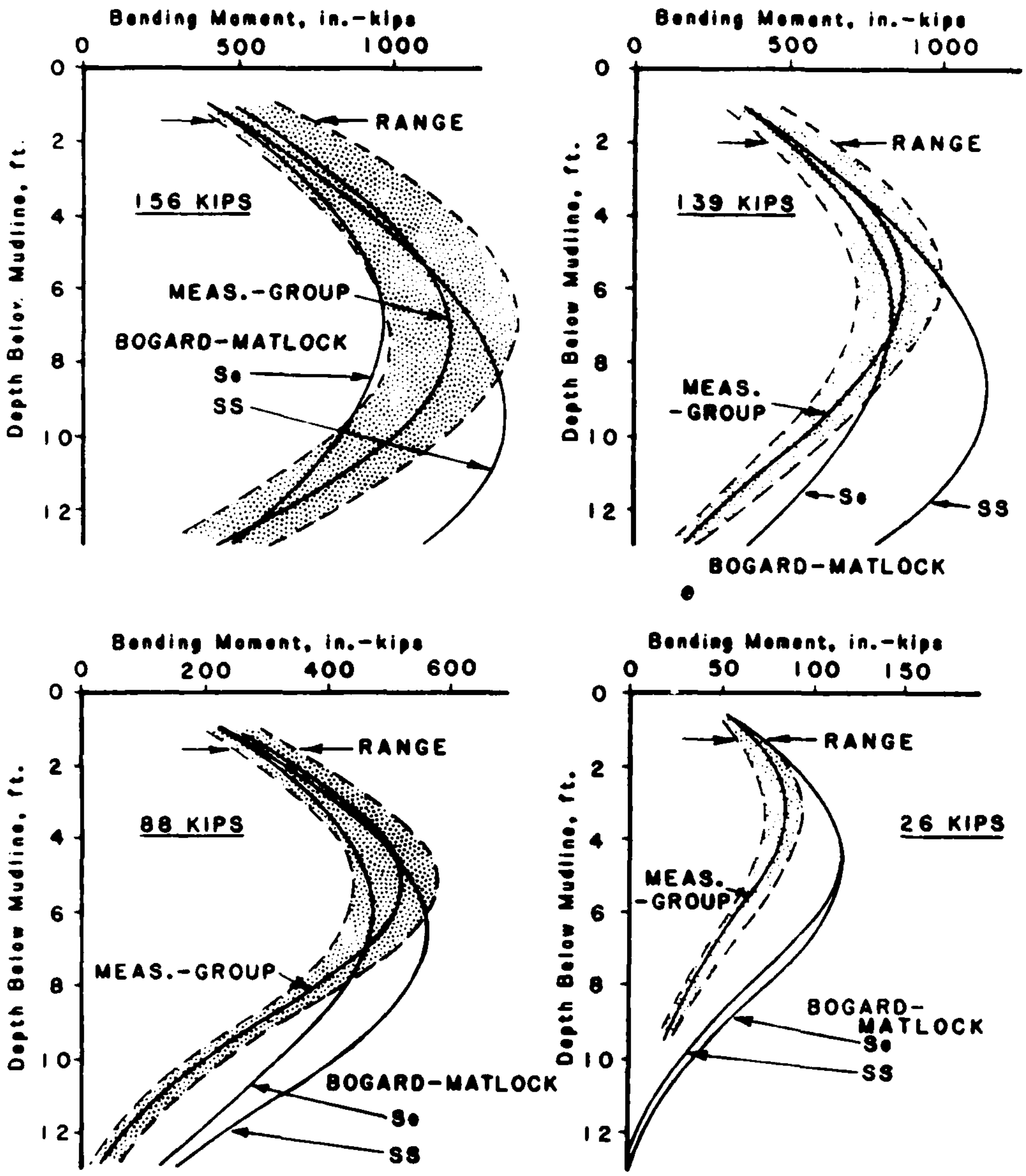


Fig. 8.27 Bogard-Matlock Bending Moment vs. Depth Predictions, Cycle 1 (Static)

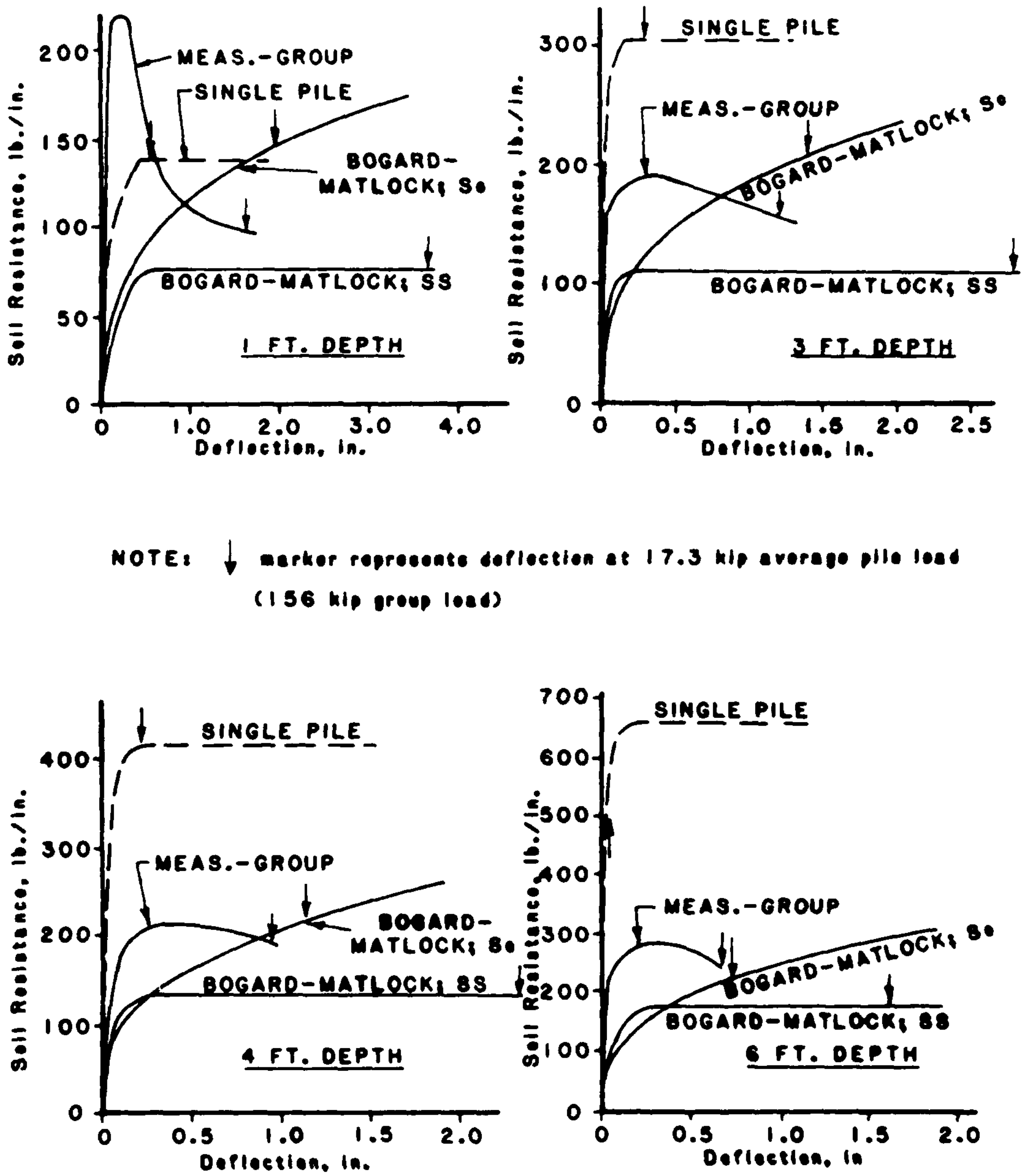


Fig. 8.28 Bogard-Matlock P-Y Curve Predictions, Cycle 1 (Static)

Cyclic Case

As with the static case the Bogard-Matlock method uses superposition of p-y curves from a single pile and from an imaginary pile to compute a predicted curve for group piles. As discussed in Chapter 4, Bogard and Matlock reduce the component of the imaginary pile for p-y curves at depths less than x_r, where x_r is related to the depth where the ground surface no longer influences the ultimate resistance of the soil. They also apparently use the static curves for the imaginary pile, thus relating all loss of soil resistance to local factors near the individual piles. These procedures were used with the SS p-y curves to analyze the behavior of the Houston group under cyclic loading. Although Bogard and Matlock proposed their method using the SO method of obtaining p-y curves, the SO method is shown not to agree with the single-pile data and therefore was not used to analyze the response of the group to cyclic loading. The SS p-y curves are of a similar form as the SO p-y curves, but the constant terms are adjusted to fit the measured single-pile response. Some adjustments were also made to force agreement in the range of cyclic degradation in soil resistance.

Load-Deflection Predictions. The predicted load-deflection response at 100 cycles is presented in Fig. 8.29. These curves show remarkably good agreement with measured response over the entire range of loads. The predictions using the Bogard-Matlock procedure for static loading were somewhat conservative, although of the proper shape.

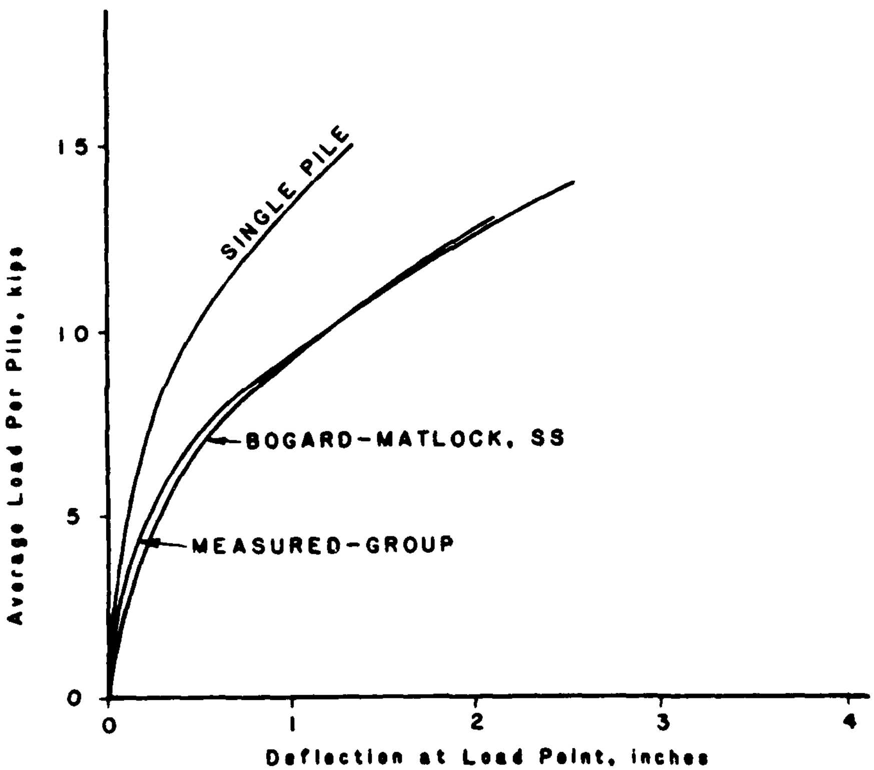


Fig. 8.29 Bogard-Matlock Lateral Load vs. Deflection Predictions, Cycle 100

Load-Moment Predictions. As revealed in Fig. 8.30, the predicted relationship between load and maximum moment is also in close agreement with measured values over the entire range of loads. In Fig. 8.31, the shape of the moment vs. depth curves is seen to match very well with measured values, even to the point of correctly predicting the depth of the maximum moments. Such agreement is encouraging in view of the magnitude of the errors that were found when using some other design procedures. This agreement is much better than that observed for the static case discussed previously.

P-y Curves. Presented in Fig. 8.32 are the computed p-y curves for cyclic loading at several depths. The Bogard-Matlock curves appear to match the curves derived from measurements fairly closely. However, the p_{ult} for the static p-y curves was underpredicted, and the curves derived from the imaginary pile were dominant. The static p-y curve for the imaginary pile dominates the shape of the predicted p-y curves here; the drop in soil resistance from some peak to a residual value is derived from the single-pile curve in this method, as is evident for the curve for the 1-ft depth. At depths of 3 ft or greater, p_{ult} from the static curve for the imaginary pile is less than the residual value of soil resistance from the single pile.

The excellent agreement that is observed is due to the fact that the values of soil resistance for the static curves were underpredicted, and these values happened to be approximately equal to the residual p values for the measured group curves. The agreement noted earlier for the predictions of load-deflection and load-moment therefore appears somewhat fortuitous. The Bogard-Matlock procedure pre-

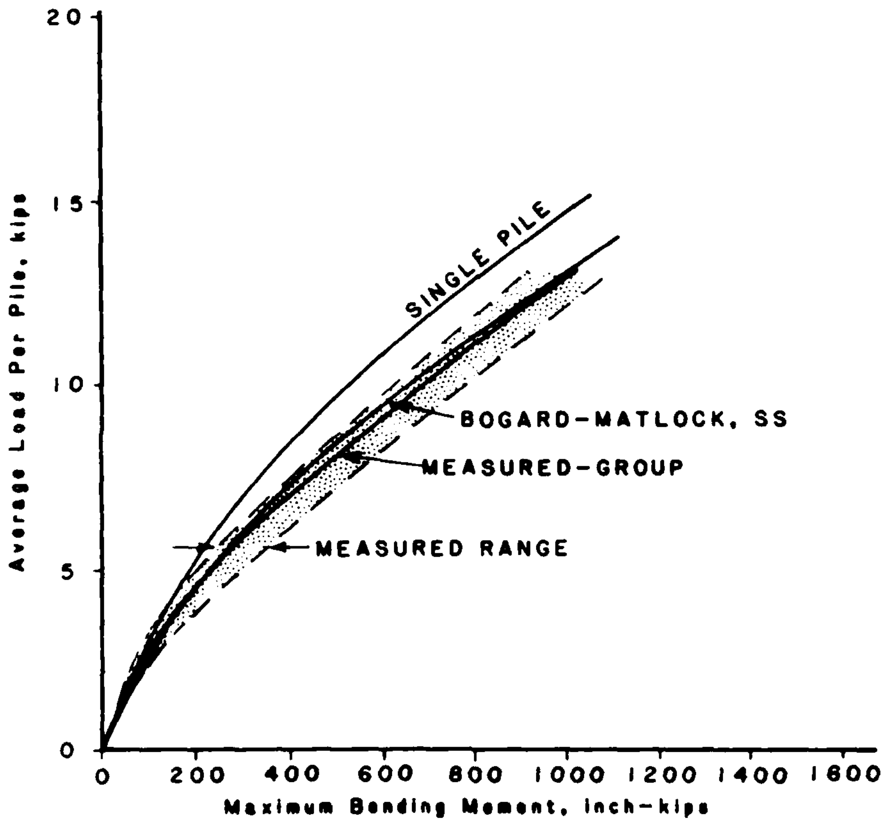


Fig. 8.30 Bogard-Matlock Load vs. Maximum Moment Predictions, Cycle 100

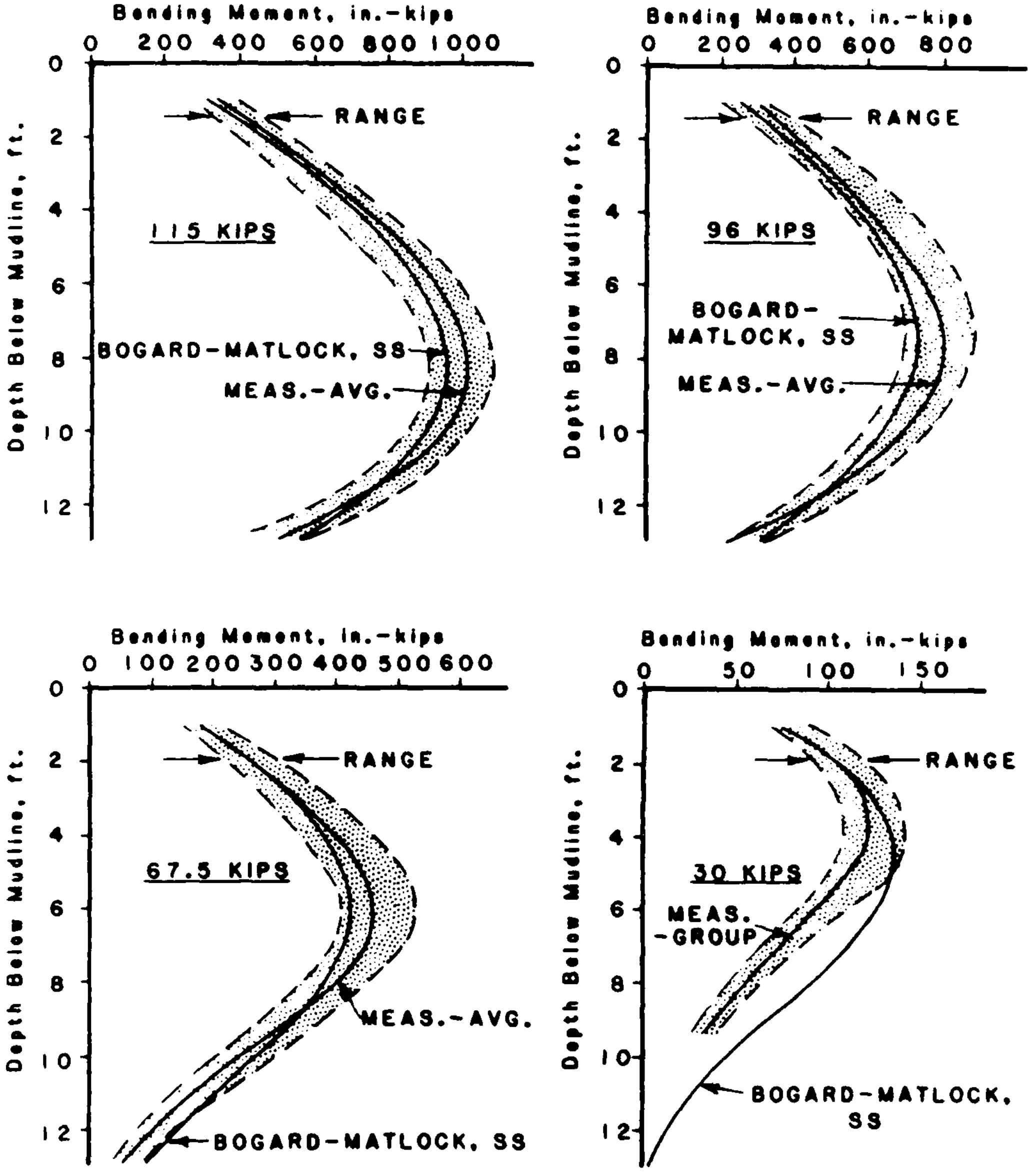


Fig. 8.31 Bogard-Matlock Bending Moment vs. Depth Prediction, Cycle 100

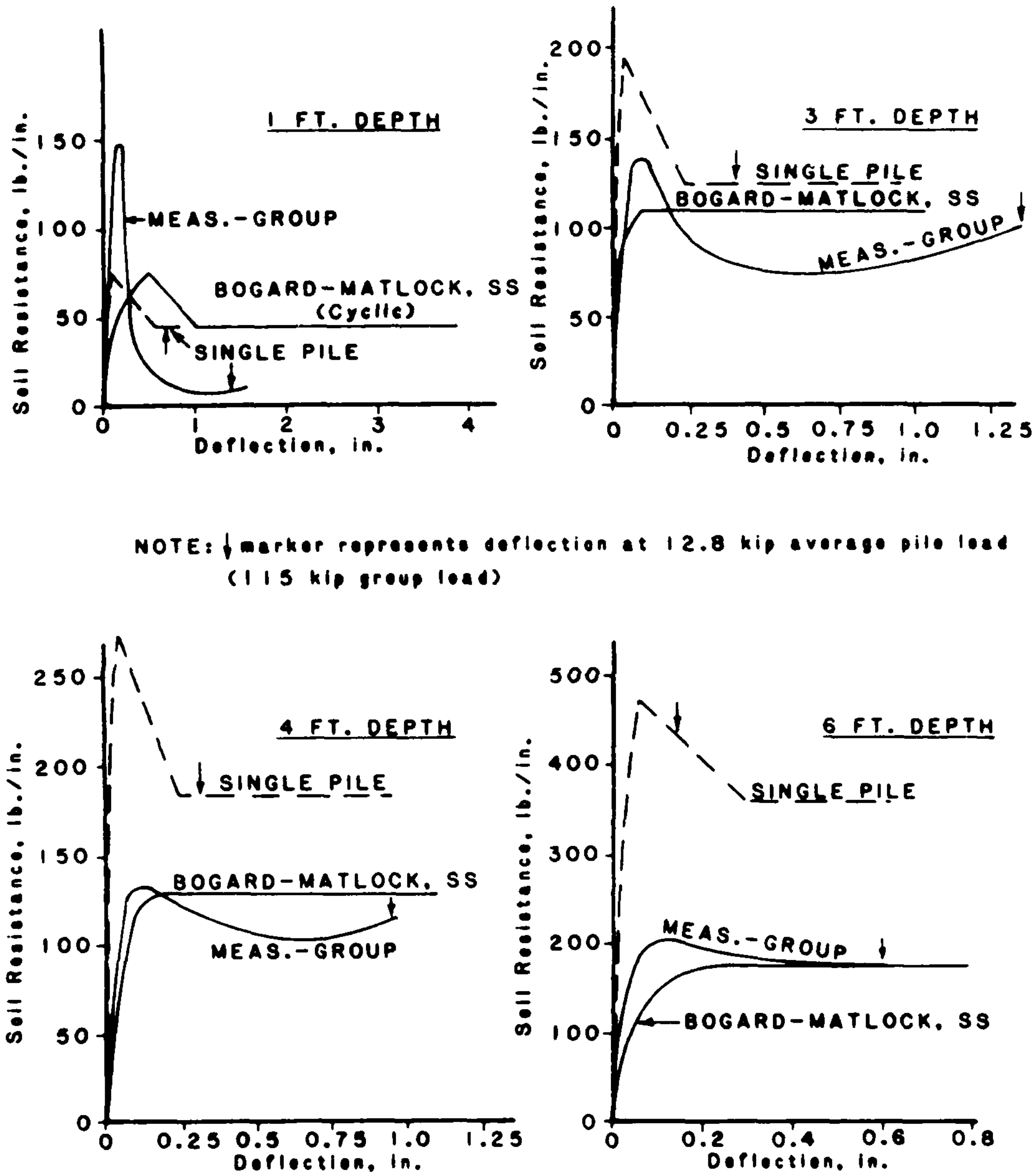


Fig. 8.32 Bogard-Matlock P-Y Curve Predictions, Cycle 100

dicts less loss of soil resistance from the static values due to cyclic loading and correspondingly predicts less increase in deflections and moments. The data shown in Fig. 8.33 illustrate this point in regard to the ratio of the residual resistance of the soil, p_{res}, to the maximum resistance of the soil from the static p-y curve, p_{ult}. With the Bogard-Matlock procedure, the static curves from the imaginary pile dominate the predicted cyclic curves at depths of 3 ft or more. Figure 8.33 makes the result of this trend clearly evident and also reveals that the measured response of the group did not follow such a pattern.

Summary

The Bogard-Matlock procedure clearly holds promise as a design tool in that it is the only one of the methods employed that provides a rational means of accounting for loss of maximum resistance of the soil due to group effects as well as due to cyclic loading. The predicted response of the group under static conditions was shown to be somewhat conservative, but the shapes of the curves showing load vs. deflection and and load vs. maximum moment were closely predicted. Observed trends such as the loss of soil resistance due to group effects and the progressive deepening of the point of maximum moment were closely predicted. The method appeared superficially to provide an even better prediction of cyclic response, but close inspection indicated this very close agreement was seen to be largely due to the conservatism in the static predictions. The degradation in response due to cyclic loading, relative to the static response, was unconservatively underpredicted. It should be noted that such a trend is

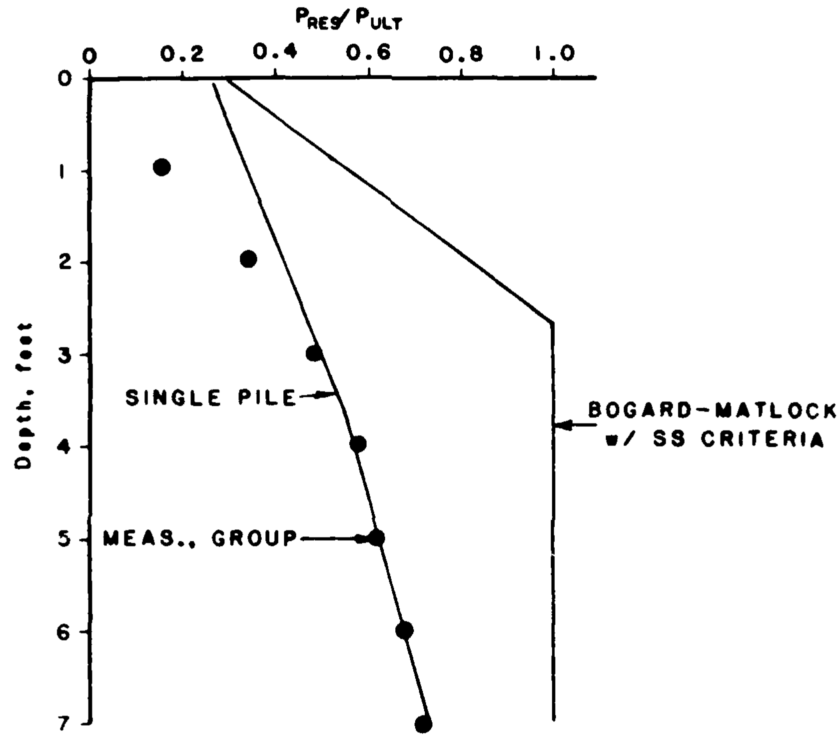


Fig. 8.33 Predicted Ratio of P_{RES} to P_{ULT} vs. Depth Using Bogard-Matlock Procedure

dependent on the method used for generating the p-y curves. The method used in this analysis fitted the single-pile response on this site; no other method would allow a simple comparison of measured vs. predicted response, as other methods would be based on an incorrect prediction of single-pile response and any agreement would be coincidental.

SUMMARY OF PREDICTIONS OF THE VARIOUS MODELS

A variety of different analytical approaches has been presented, and a variety of resulting predictions has ensued. In summarizing, it can be concluded that the elasticity-based approaches, including the hybrid models, do not predict the nonlinearity of the response to an adequate degree. These models underpredict the deflections, moments, and depth to maximum moment at loads exceeding small values. Although these models were never offered as adequate at loads exceeding working loads, how might one define working-load values if the ultimate loads are unknown? The failure of these approaches to model behavior at large loads is attributable to the lack of any rational means of reducing the values of the ultimate resistance of the soil to account for group effects. The fact that distribution of loads to piles in the group did not follow the predictions of the elasticity theory also does not inspire confidence in an elasticity-based approach.

On behalf of the elasticity-based models (such as DEFPIG and PIGP2R), it must be stated that none of the other types of procedures can account for varying pile arrangements and general loadings (i.e.,

coupled axial and lateral loads). A designer faced with a group geometry and/or loading which cannot be represented by a simplified approach presently has little alternative to the use of an elasticity-based model; the data presented in this chapter offer some guidance in determining the limitations of such an approach.

The model proposed by Bogard and Matlock must be said to show promise, in that it correctly predicted trends of nonlinearity in the group response. It also provided a means of reducing values of soil resistance to account for group effects, which seems to be the key to accurate predictions of response. The most glaring shortcoming of the Bogard-Matlock method is the empiricism used in constructing p-y curves for the pile group and its inability to predict different behavior in different piles. Such techniques as dividing deflection values on the imaginary p-y curve by the pile spacing in diameters appear to be largely the result of forcing a fit with the Harvey-group tests on which the method is based. Although Bogard and Matlock do not claim their method to be a rigorously correct solution, the application of such an approach to design problems is extremely sensitive to calibration of the procedure with experimental data. Of course, none of the other proposed methods have been calibrated to test data either, so the demand for more experimental data seems to be a universal requirement.

With regard to cyclic loading, all of the models that can account for cyclic degradation in soil resistance assume the residual soil resistance to be close to that of the single-pile value, which in fact was clearly not the case. The Focht-Koch procedure has a problem

even modelling the loss of soil resistance to this value, because stretching the near-surface p-y curves with a Y-multiplier prevents the residual part of the curve from ever being reached. Although the Bogard-Matlock procedure seemed to predict cyclic response very well, this was shown to be partly due to the fact that the method conservatively predicted static response; cyclic degradation with respect to the static response was underpredicted for this method as well as for all of the others.

The observation that the residual values of soil resistance for the cyclic p-y curves were less than that of the single pile would seem to indicate that there was some loss in soil resistance from cycling due to effects other than gapping or scour immediately around the individual piles. Perhaps a pore-pressure buildup occurred in the mass of soil around the group, reducing the undrained shear strength somewhat. One can only speculate, since such measurements were not made. It must be concluded, however, that some means of accounting for loss of soil resistance to values less than that of the single-pile values is needed, and none of the available models make such an accounting.

CHAPTER 9

RECOMMENDATIONS FOR DESIGN

INTRODUCTION

The previous chapter has provided some insight into the suitability of currently available analytical models for use in design. Comparison of predicted vs. measured results emphasized the highly nonlinear behavior of the group during lateral loading. None of the elasticity-based models adequately predict this behavior and in fact led to unconservative predictions of deflection and pile stresses. A comparison of measured soil resistances for the group piles and the single piles revealed a substantial reduction in the ultimate soil resistance for the group relative to that of the single pile.

The single-pile method and the Bogard-Matlock method offer a seemingly rational approach to predicting load-transfer relationships for piles in a group relative to that of a single pile, at least for static loading. Although the Bogard-Matlock procedure attempts to account for the effects of cyclic loading, the predictions of cyclic behavior for the Houston pile group was seen in Chapter 8 to be unconservative.

A simple procedure for predicting the response of groups of closely spaced piles in clay subjected to static or cyclic lateral loading is outlined in this chapter. The procedure is not offered as a rigorous analytical model, but rather as an interim procedure which

has a rational basis and which is correlated with experimental data. The method is simple and utilizes existing design methodology (the p-y concept). The paragraphs that follow describe the proposed method and discuss the application of the method in design. Predicted results of the Houston, Texas, and Harvey, Louisiana, experiments are examined.

THE P-FACTOR APPROACH

An examination of measured p-y curves for the Houston group and single piles reveals the following:

1. The ultimate soil resistance (p_u) for static loading of an average pile in the group is substantially lower than that of the single pile, and the difference increases with increasing depth.
2. The soil resistance after cyclic loading at large loads is also less for the group piles relative to the single pile.
3. The deflection required to mobilize the ultimate soil resistance (or residual soil resistance in the case of cyclic loading) can be treated as if they are approximately equal to corresponding deflections for the single-pile p-y curves.

These observations lead to the speculation that an approximation for the average p-y curves for the group might be made by "scaling down" the single-pile p-y curves with some factor applied to the soil resistance. A reasonable factor is the ratio of ultimate soil resistance (p_u) during static loading of an average group pile to that of a single pile. Ratios of the average measured p_u for group piles to

those of the single pile are plotted vs. depth in Fig. 9.1. A relatively smooth curve of a decreasing ratio with increasing depth is observed.

As an attempt to determine whether or not such an approach could be rewarding, computations were performed for conditions of the Houston experiment using p-y curves from the single-pile experiment that were "factored." This factoring was performed by multiplying the soil resistance at each point on the p-y curve at each depth by the ratio shown in Fig. 9.1. This procedure was followed for both the static and cyclic curves, as demonstrated in Fig. 9.2. Single-pile p-y curves deeper than 6 ft had to be estimated since measured curves were not obtained at great depths. Curves deeper than 8 ft were factored by a ratio of 0.3. Several typical curves are shown in Figs. 9.3 and 9.4. The predicted and measured curves should match at the last point on the measured curve, as that point was used to compute the ratios shown earlier. Cyclic p-y curves are predicted reasonably well with this approach, with increasing conservatism with depth.

Presented in Figs. 9.5 through 9.10 are plots of computed and measured deflections and moments as a function of load along with moment vs. depth curves at several loads. Of particular importance in design are the predictions of maximum bending moment.

The results discussed above lend encouragement to the "p-factor" approach, provided that some reasonable means of estimating these multipliers can be found. The following section examines the problem of predicting this reduction in soil resistances.

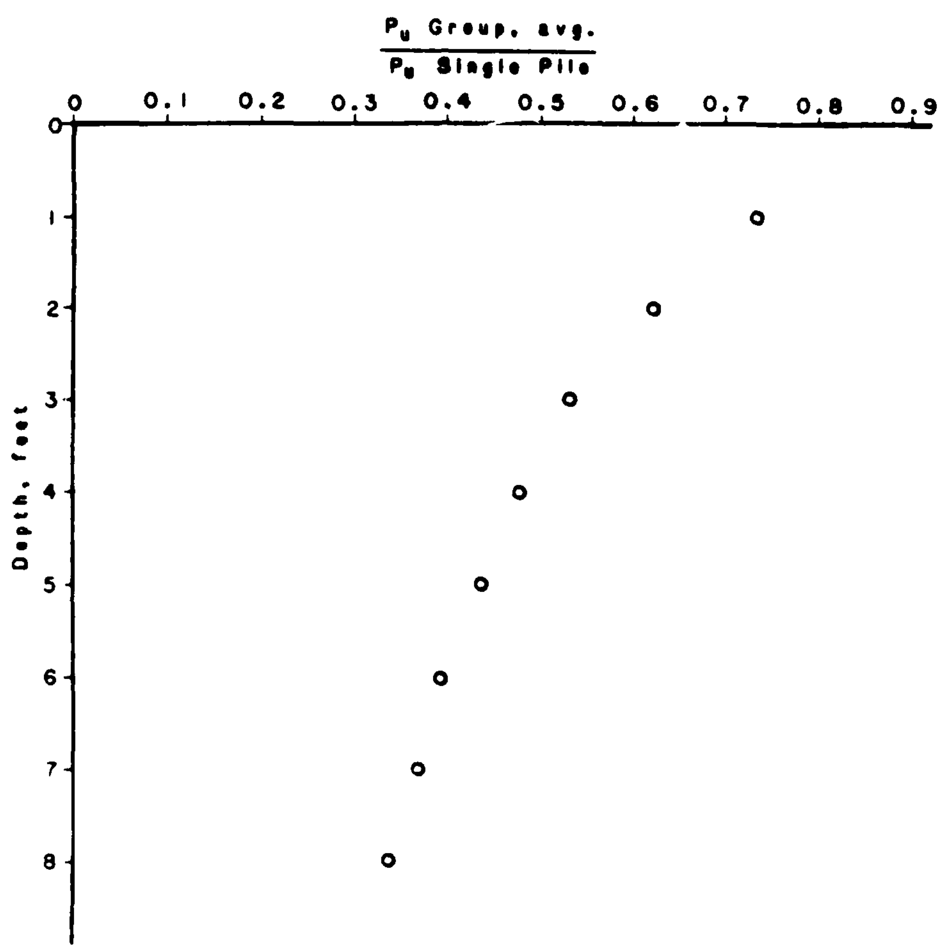


Fig. 9.1 Measured P_u Values for Group, Relative to Single Pile Values

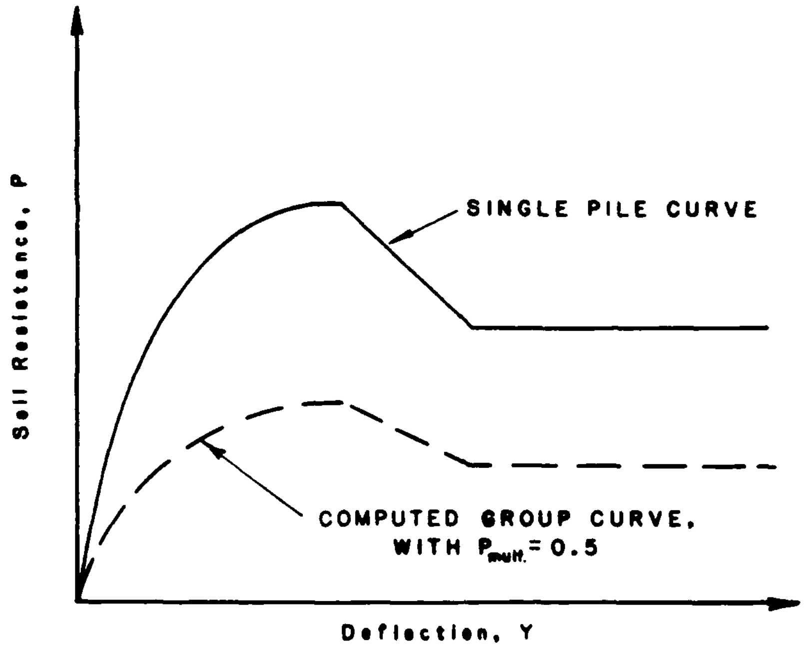


Fig. 9.2 Interim Procedure for Computing Average Group Pile P-Y Curve

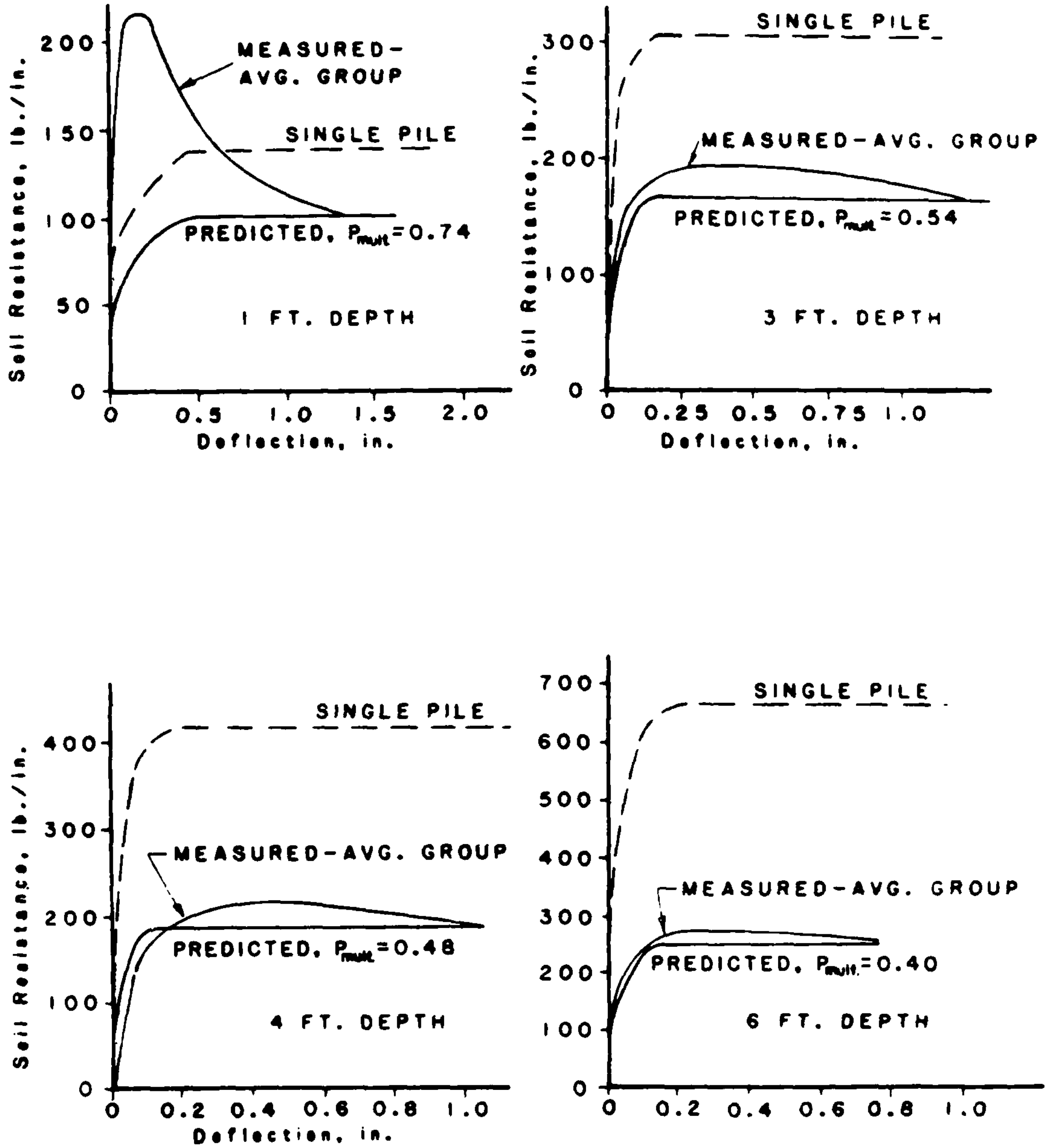


Fig. 9.3 Predicted Static P-Y Curves for Group with Measured P-Factors, Houston Site

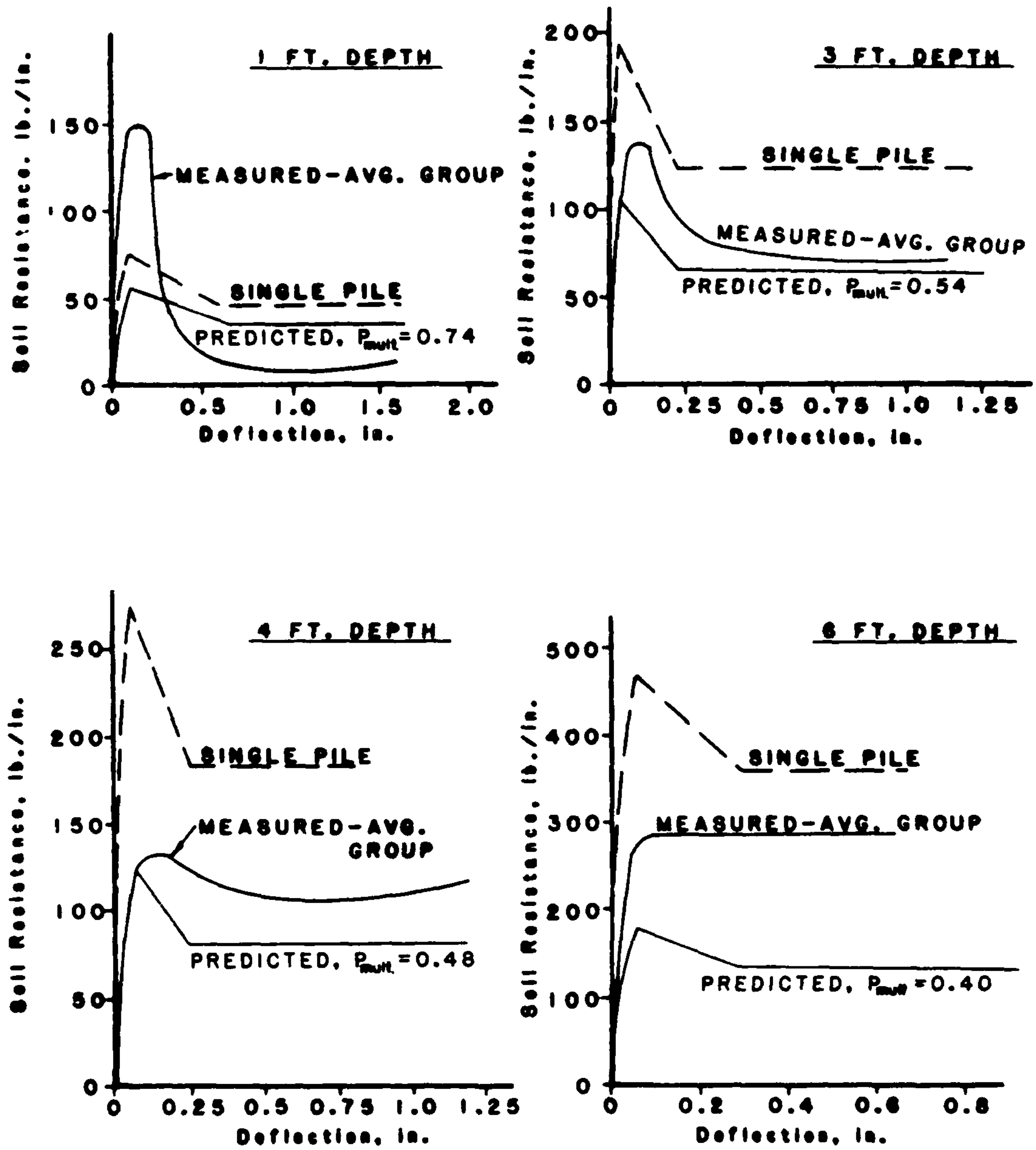


Fig. 9.4 Predicted 100 Cycle P-Y Curves for Group with Measured P-Factors, Houston Site

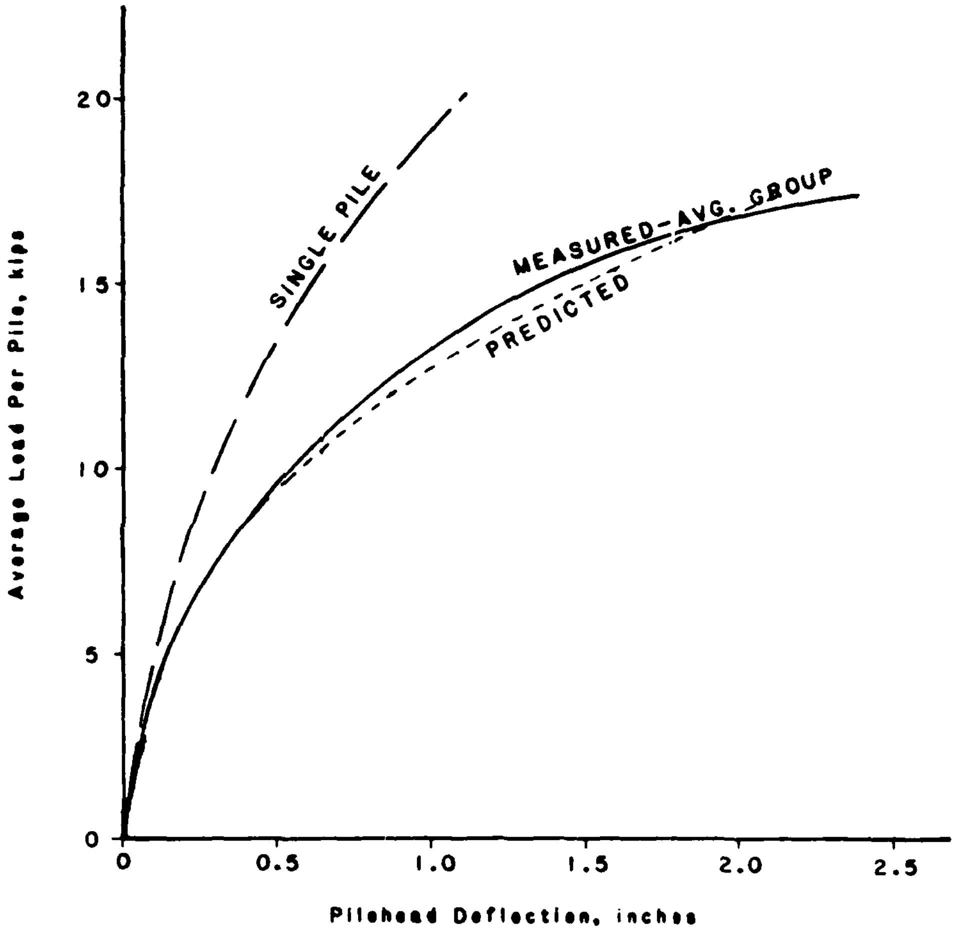


Fig. 9.5 Predicted Lateral Load vs. Deflection, for the Houston Site using Measured P-Factors, Cycle 1 (Static)

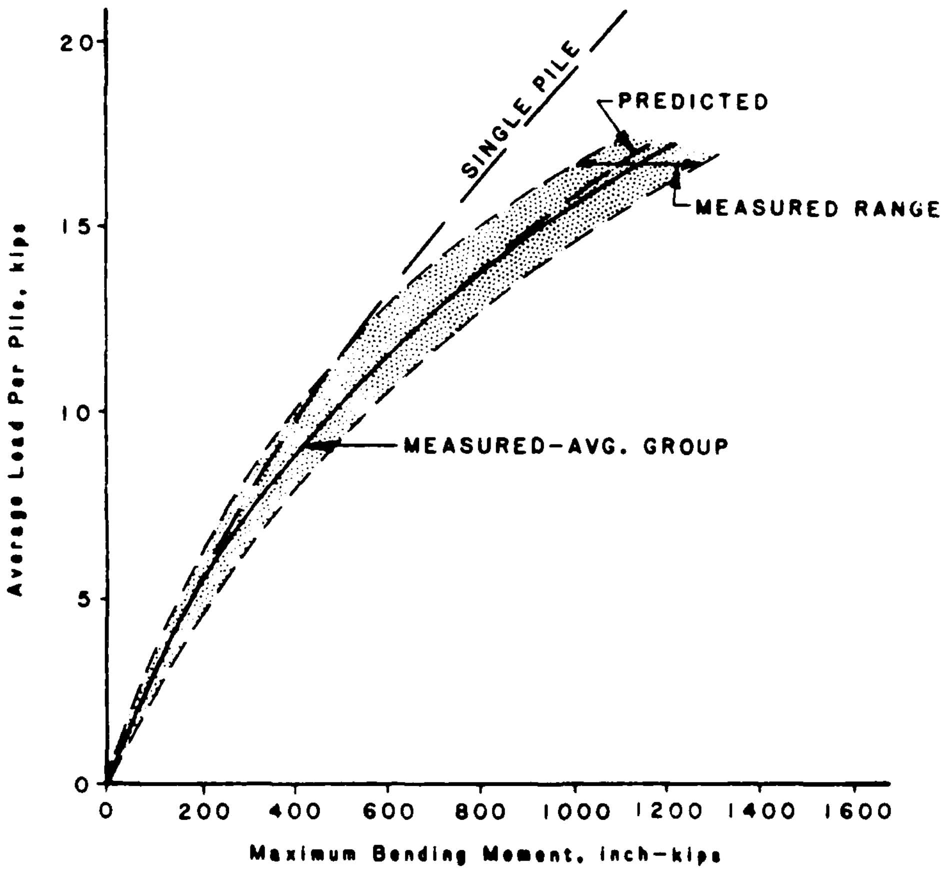


Fig. 9.6 Predicted Load vs. Maximum Moment for the Houston Site using Measured P-Factors, Cycle 1 (Static)

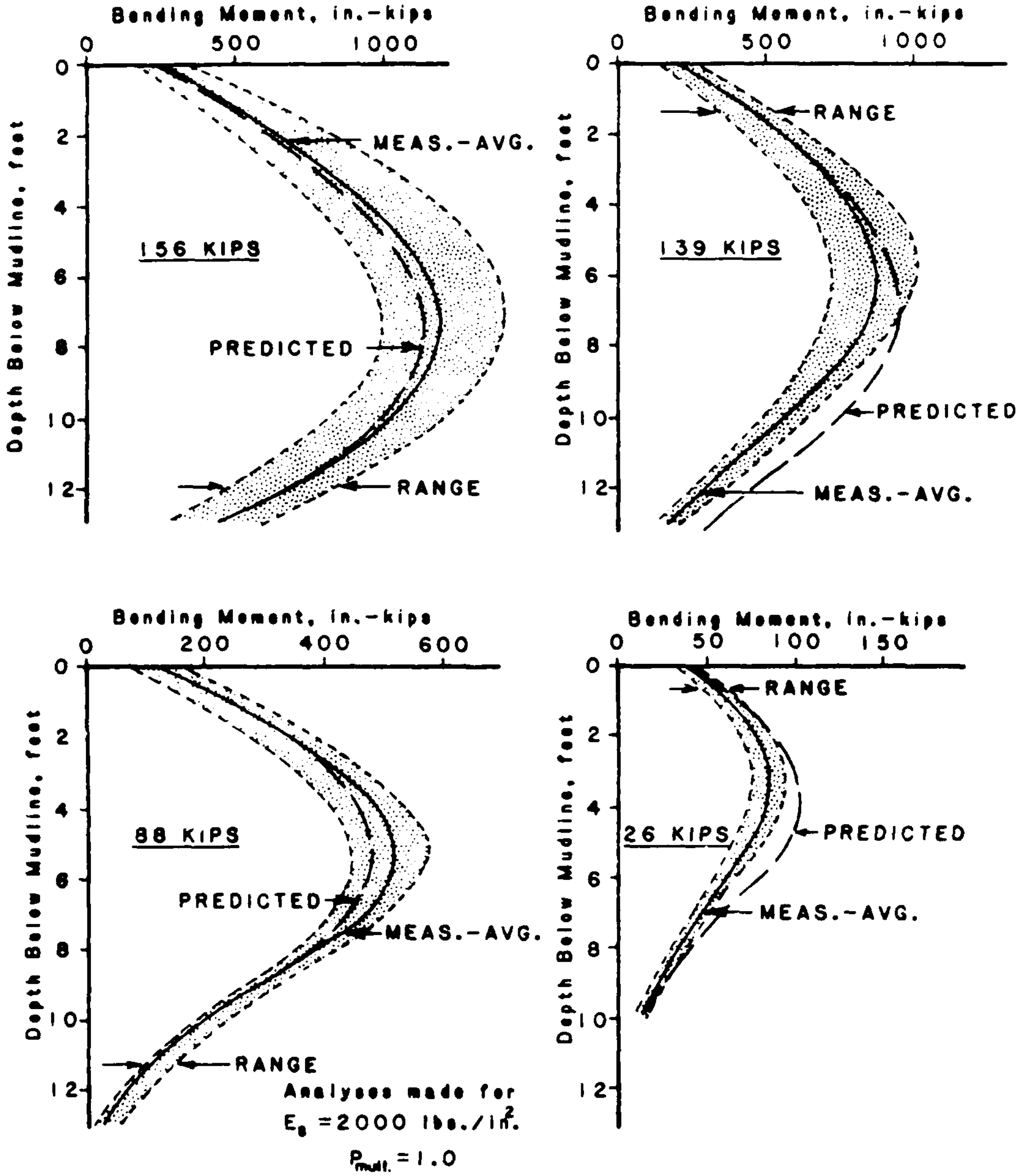


Fig. 9.7 Predicted and Measured Bending Moment vs. Depth, Cycle 1 (Static)

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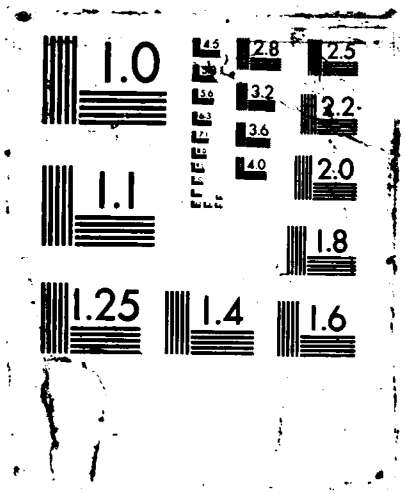
1.0
1.1
1.25
1.4
1.6
1.8
2.0
2.2
2.5
2.8
3.2
3.6
4.0

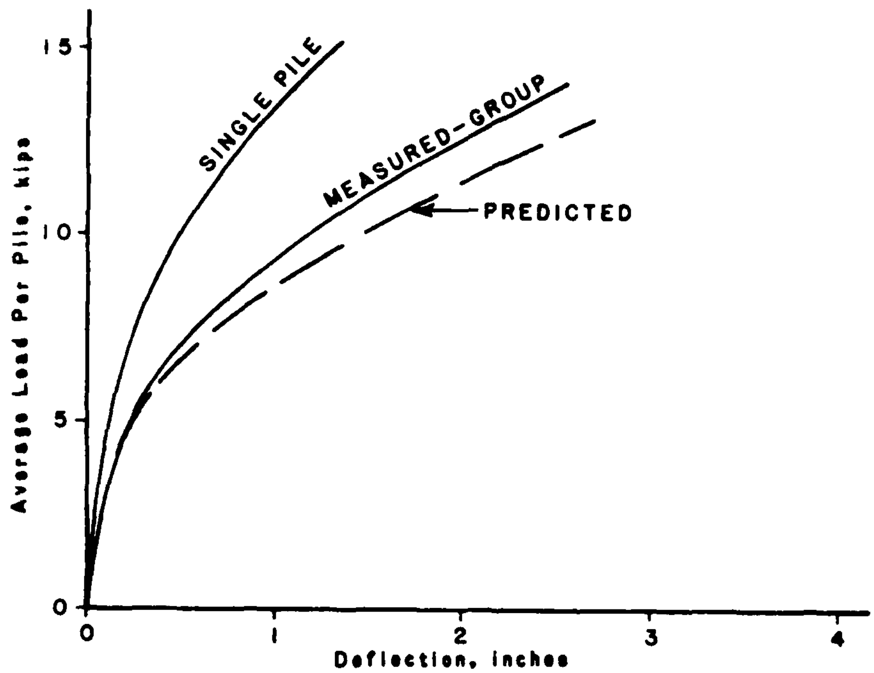

Fig. 9.8 Predicted Load vs. Deflection for the Houston Site using Measured P-Factors, Cycle 100

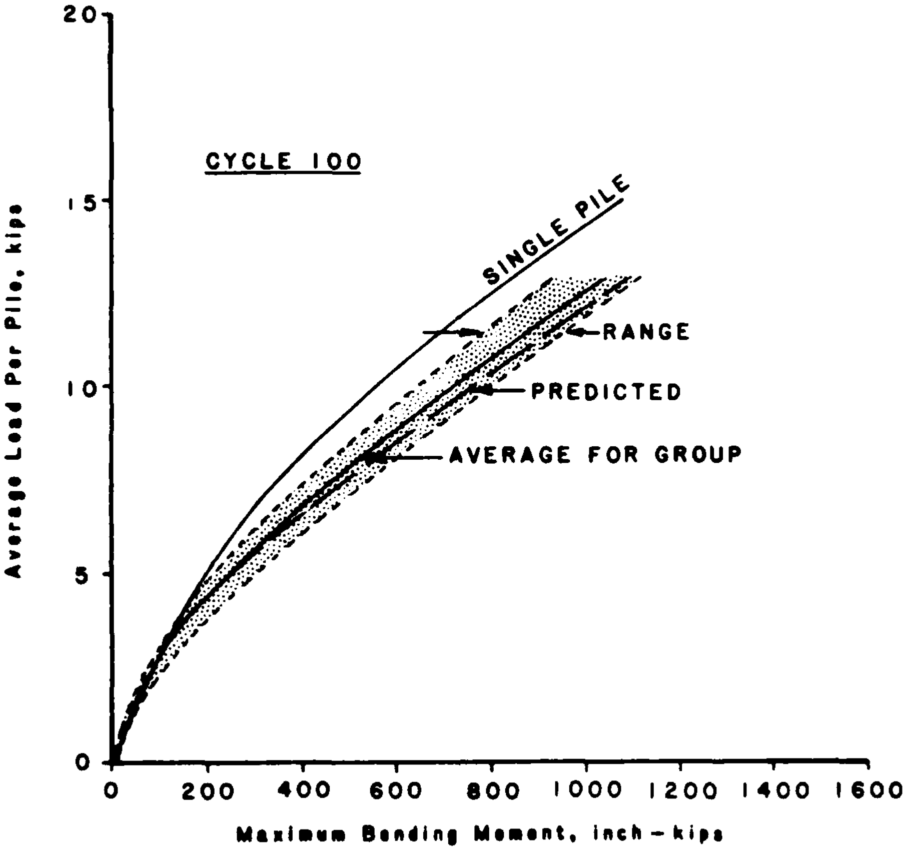

Fig. 9.9 Predicted Lateral Load vs. Maximum Moment for the Houston Site using Measured P-Factors, Cycle 100

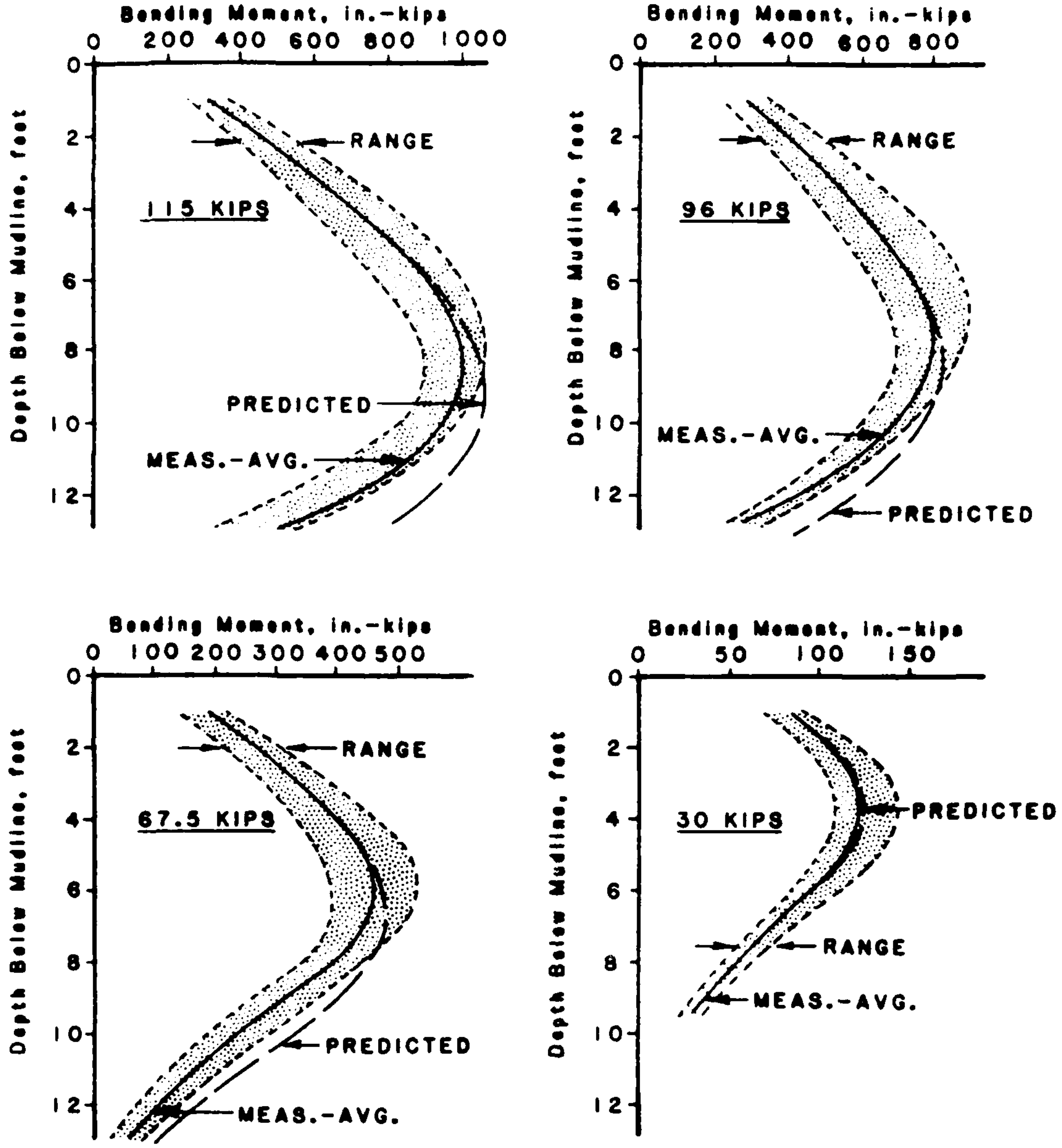

Fig. 9.10 Predicted and Measured Bending Moment vs. Depth, Cycle 100

DETERMINATION OF P-FACTOR

The single-pile method as well as the design method offered by Bogard and Matlock provide a seemingly rational way of predicting a reduced maximum soil resistance for piles in a group relative to that of a single pile, at least for static loading. Both of these methods limit the cumulative soil resistance of all the piles within a group to that of a single, large-diameter pile. Assuming that it is reasonable that a group of piles will act like a single, large-diameter pile, one is then faced with the problem of predicting the soil resistance of a large-diameter pile. Although existing p-y criteria have been used to make such a prediction, the effect of diameter on soil resistance varies considerably with different criteria.

As an example of the potential difference in estimating ultimate soil resistance in the group with the large pile and with differing p-y criteria, the ratio of group to single-pile p_u values vs. depth for the Houston site by several methods is presented in Fig. 9.11. Predictions of ultimate soil resistance for piles in the group are based upon that of a pile 85 inches in diameter (which has a circumference equal to that of the Houston group) divided by 9 (the number of piles in the Houston group). Points are plotted to represent measured values. The line marked "SO" represents the soft clay criteria (with the empirical term J set equal to 0.25) as used by Bogard and Matlock in their analysis of the Harvey test. It should be recalled that the Harvey test piles were not sufficiently instrumented to derive p-y curves. Bogard and Matlock used p-y curve criteria for soft clay tailored to produce a bending moment vs. depth curve which

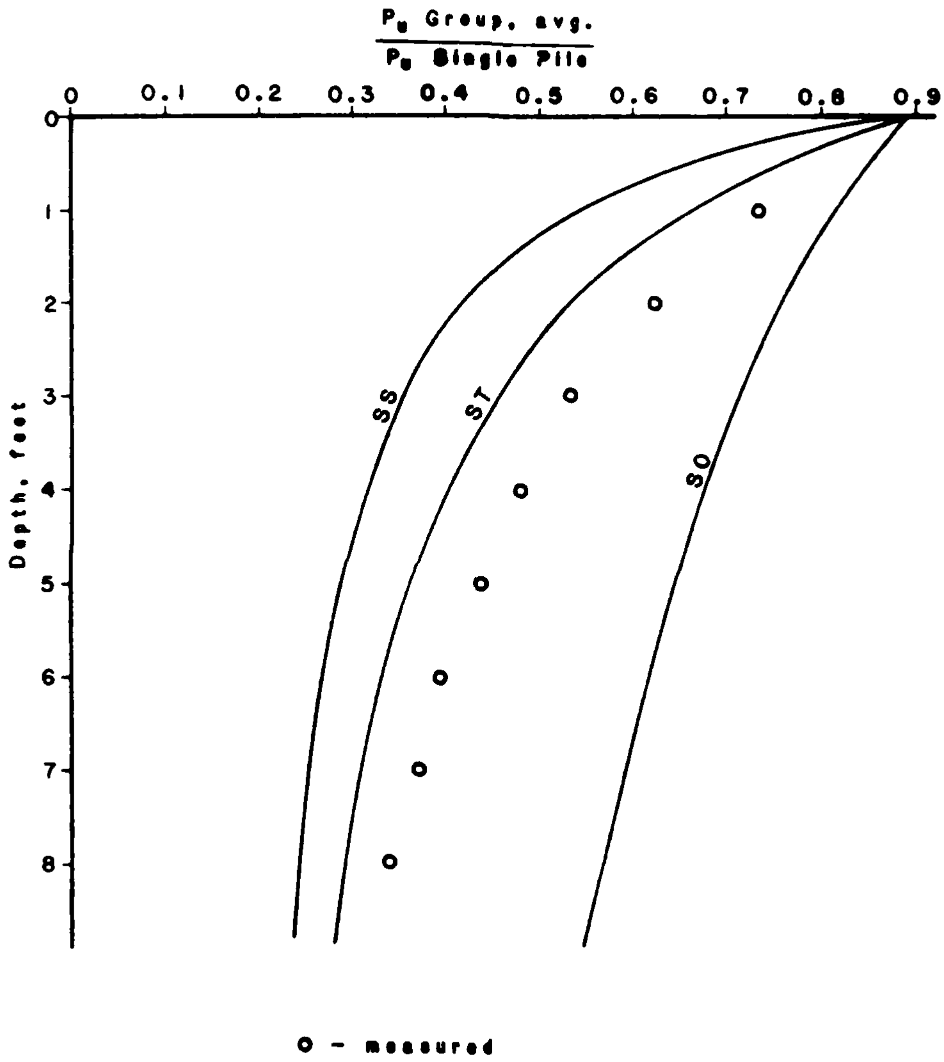

Fig. 9.11 Predicted and Measured P_u Values for Group, Relative to Single Pile Values (Houston Site)

approximately fit the measured bending moments. The line marked "SS" resulted from the extrapolation of the criteria fitted to the measured single-pile data, as described in Chapter 8.

Considering the wide scatter in measured undrained shear strength (S_u), it is possible that reasonable S_u values could be back-calculated such that the p_u values from the SO criterion would match the single-pile test. With those values of S_u and the SO criterion, the group pile p_u values from the large pile technique would be greatly overestimated. A similar approach using the SS criterion would result in p_u values for the group piles which would be greatly underestimated.

Another suggestion for ultimate static soil resistance at shallow depths was originally proposed by Reese, Cox, and Koop (1975) and extended by Sullivan (1977) where:

$$p_u = 2S_{ua}b + \gamma'b + 0.83S_{ua}x$$

where: S_{ua} = average undrained shear from the surface to depth x,

γ' = effective soil unit weight, and

x = depth.

This relationship was used to develop the "Unified Method" by Sullivan (1977). This criterion was used for the Houston site with the term "F" assumed equal to 1.0 (see Reese (1984) for a brief description of this method). Computation of p_u for the group piles using the large pile approach results in the line marked "ST" in Fig. 9.11. The ST curve is seen to more nearly match the measured values; however, this figure is not offered as evidence that the ST relationship for p_u is

any more nearly correct than any other, only that it matches the measured data better with the large-pile procedure. It should be noted that the Reese, Cox, and Koop recommendations for static p-y curves in stiff clay below the water table suggest a precipitous reduction in soil resistance after some early peak value. Although some reduction in soil resistance at shallow depths might be expected as deeper failure zones are mobilized, the large reduction in the Reese, Cox, and Koop recommendation is widely acknowledged to be the result of fitting that procedure to a test in heavily fissured clays (Manor, Texas) and is not considered to be typical of most stiff clays. Note also that the three curves shown in Fig. 9.11 are <u>not</u> unique, but depend upon the shear strength vs. depth profile. A unique set of curves would be indicated for these criteria for any assumption of constant shear strength with depth, but such was not the case at the Houston site.

The concept of the limiting soil resistance of a large imaginary pile does not directly address the problem of loss of soil resistance during cyclic loading. The gapping and scour around the group piles due to cyclic loading was observed to be restricted to the near-field around the individual piles both in the Houston test and in the 5-pile group test in Harvey, Louisiana. Bogard and Matlock proposed that the residual soil resistance for cyclic p-y curves should be near that of a single pile; measurements on the Houston group demonstrated that this residual value of soil resistance was lower for the group than for the single pile. Figure 7.31 indicated that p_{res}/p_{ult} for the average group pile was almost identical to the ratio for the single pile.

PROPOSED DESIGN METHOD

In light of the above discussion, a complicated procedure for accounting for group effects does not seem justified. Spatial variation in resistance within the piles in the group cannot presently be accounted for, and the use of an "average" p-y curve for all piles in the group must be considered an approximation. The most important considerations in design include:

1. a close approximation of pile stresses,
2. reasonable confidence in the method of accounting for cyclic degradation in soil resistance, and
3. an approximation of deflection and rotation.

The Bogard-Matlock approach is seen as an attempt to address these considerations. However, the combination of a single-pile and a large-pile p-y curve (with adjustment for deflection in the large-pile curve) was found to overpredict soil resistance during cyclic loading for the Houston group. Additionally, the calculations involved in the various steps in the Bogard-Matlock approach was found to be tedious and complex, especially in light of the lack of experimental evidence to support such an approach. In consideration of the amount of experimental data and the nature of the pile-group problem, the following method is proposed for obtaining the response of the piles in a group.

1. Compute a set of p-y curves for a single pile using judgement as to the best available p-y criteria for the site; the "Unified Method" (Reese, 1984) is recommended.
2. Using the same p-y criteria, compute a limiting soil resistance for the average pile in the group based upon a

large imaginary pile having a circumference equal to the perimeter of the group and on the relationship:

$$p_{ug} = \frac{1}{n} p_{ui}$$

where: p_{ug} = ultimate soil resistance of the average pile in the group,

p_{ui} = ultimate soil resistance of the large imaginary pile, and

n = number of piles in the group.

3. Compute a p-multiplier, p_{mult}, by:

$$p_{mult} = p_{ug}/p_{usp}, \quad p_{mult} \leq 1.0$$

where: p_{usp} = ultimate soil resistance of a single pile, computed in step 1.

4. Adjust the single-pile p-y curves for group effects by multiplying each value of soil resistance by p_{mult}. This procedure is used for both static and cyclic p-y curves, as shown in Fig. 9.2.

The proposed method is offered only as an interim method to account for group effects and is based on test results available to date; additional analytical and experimental studies are certainly needed. As has been discussed, the computed p-y curves are very sensitive to the interpreted shear strength and the relationship chosen between shear strength and ultimate soil resistance. As will be discussed in the paragraphs to follow, the method has only been shown to produce acceptable results using p_{mult} factors determined with the "Unified Criteria" as described by Reese (1984). A summary of the proposed

method is provided in Appendix A. Computed pile group behavior using this method for the Houston and Harvey experiments follow.

It should be noted that the soil resistance during cyclic loading of a single pile at the Houston site is underpredicted by the Unified Method. Although some modifications of the Unified Method for cyclic loading may be in order, this subject is left for future research. The cyclic residual soil reistance (i.e, the soil resistance after many cycles of load) in the discussions which follow has been based upon results of the single-pile results at the Houston and Harvey sites, so as to allow a more proper comparison of group and single pile behavior.

HOUSTON TEST

In order to establish a correct prediction of single-pile behavior, a smooth approximation to the measured single-pile curves was used. The ultimate soil resistance, p_u was assumed to match the minimum of the following relationships:

$$p_u = \left(2 + \frac{\bar{\sigma}_v}{S_{ua}} + 0.833\,\frac{x}{b}\right) S_{ua} b \tag{9.1}$$

$$p_u = \left(3 + 0.5\,\frac{x}{b}\right) S_u b \tag{9.2}$$

$$p_u = 9 S_u b \tag{9.3}$$

where: x = depth,

b = pile diameter,

$\bar{\sigma}_v$ = effective overburden stress at depth x,

S_u = undrained shear strength at depth x, and

S_{ua} = average undrained shear strength from the ground surface to depth x.

In order to examine the accuracy of the design method for pile groups, it is important to separate errors in predicting the response of a single pile from those errors introduced in accounting for group effects. For this reason, values of S_u were backfigured for each equation such as to match the interpreted p_u from measurements at each depth; the maximum S_u backfigured from the three equations was used as the correct value of S_u. A bilinear approximation to the backfigured S_u vs. depth relationship was used in subsequent analyses and is shown in Fig. 9.12. Because this relationship seems reasonable given the scatter in the shear strength data, it was accepted for computational purposes. Equation 9.1 governed behavior in the upper 7 feet. Below that depth, measured p_u values from the single-pile test could not be ascertained; the shear strength interpretation below 7 ft is based primarily on UU triaxial compression test results.

Using the three equations described previously, a p_u vs. depth relationship was computed for a single pile (p_{usp}) and a large imaginary pile (p_{uip}) using b = 85 inches. The p_u values for an average group pile, p_{ugp}, were then taken as

$$p_{ugp} = \frac{1}{n} p_{uip}$$

where: $p_{uip} = p_u$ for b = 85 in., and

n = number of piles in group = 9.

Shown in Fig. 9.13 are p_u vs. depth predictions for the average group pile and single pile, along with measured average values for the Hou-

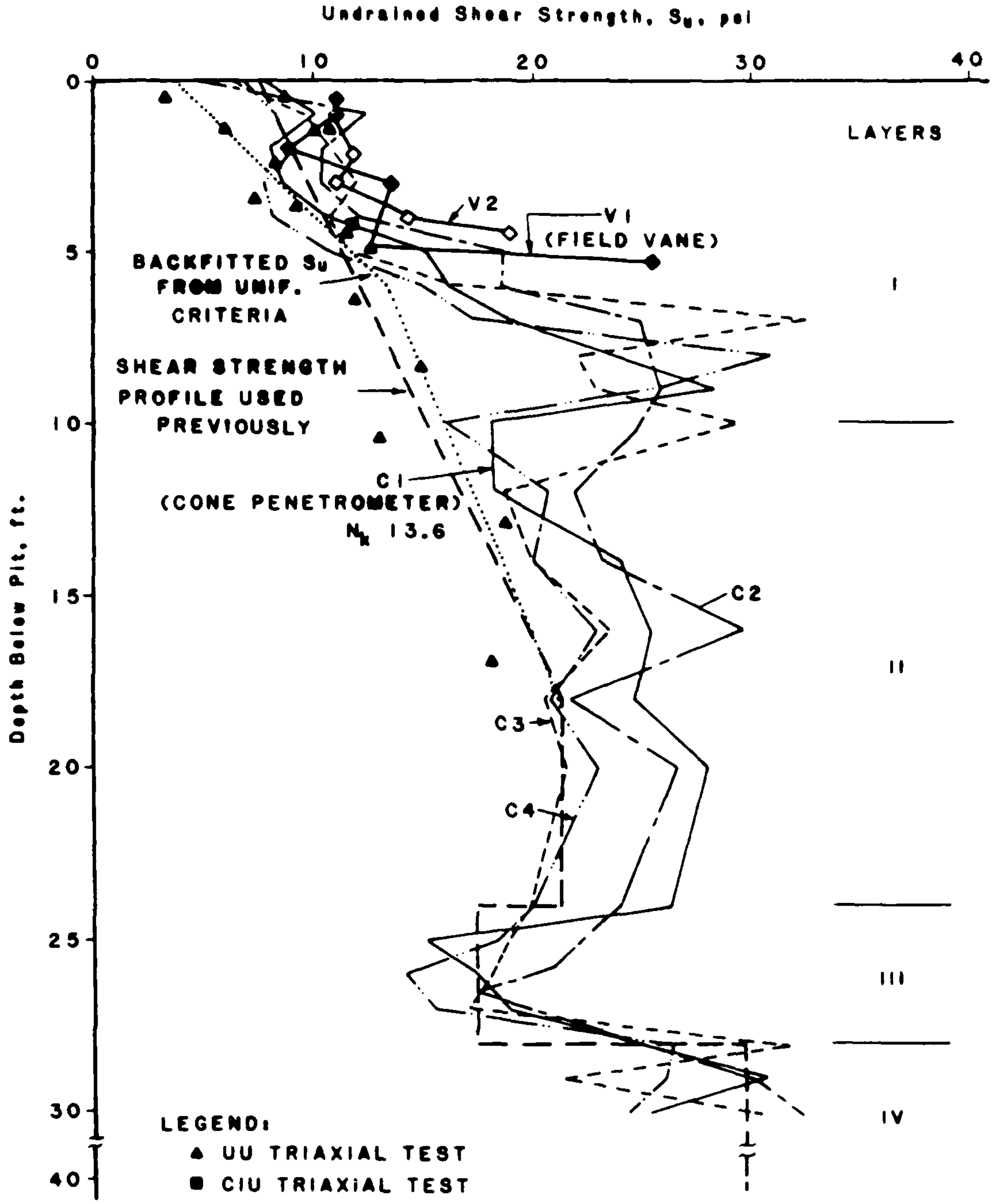

Fig. 9.12 Undrained Shear Strength vs. Depth

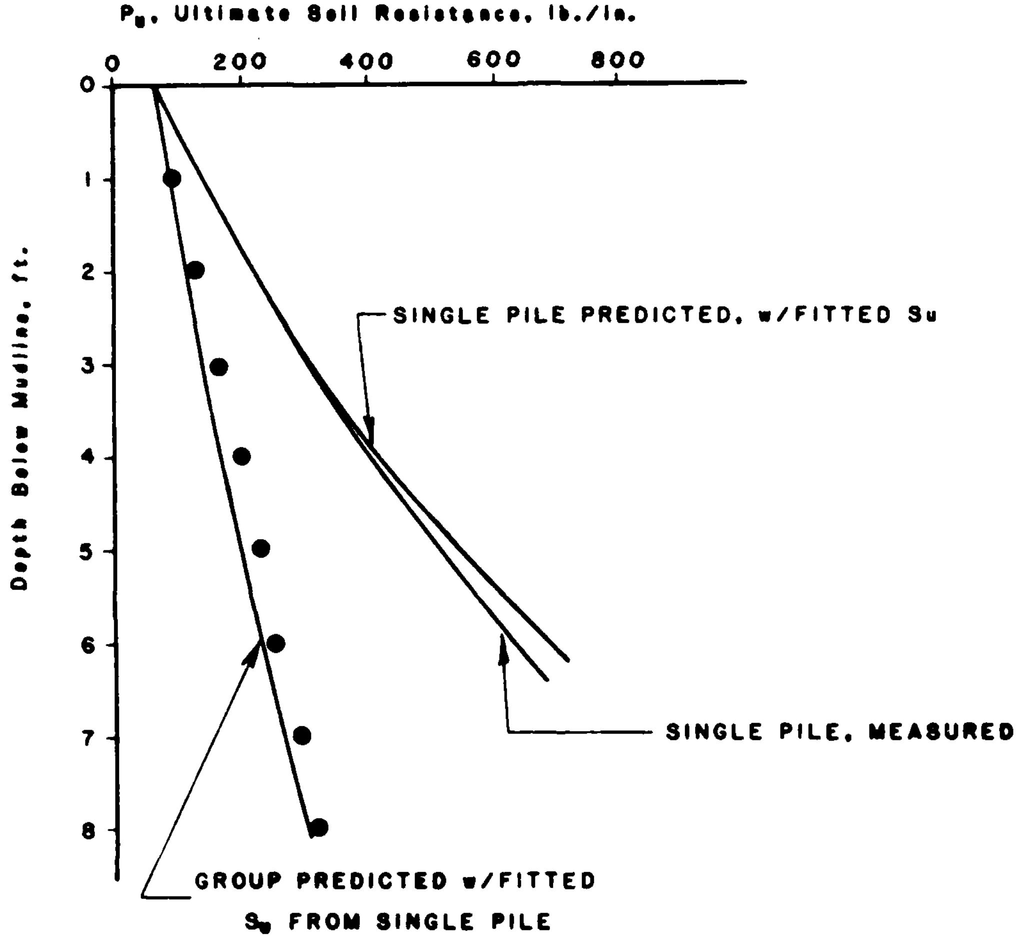

Fig. 9.13 Predicted and Measured P_u vs. Depth Relationship

ston group (measured p_u for the group was taken to be the soil resistance at large deflection as described in Chapter 7, rather than as peak value). Fairly good agreement is observed. P-multiplier values (p_{mult}) are computed from the ratio of the p_u values for a group pile to single pile: $p_{mult} = p_{usp}/p_{ugp}$. Values of p_{mult} for this case are eoual to those shown by the curve marked "ST" in Fig. 9.11.

The p-y curves for the group for both the static and cyclic cases were computed by multiplying the soil resistance values on the single-pile p-y curves by the p_{mult} factor at each depth considered. Several p-y curves computed by this method are given in Figs. 9.14 and 9.15. These curves generally indicate somewhat conservative predictions of soil resistance, particularly for cyclic loading.

Presented in Figs. 9.16 through 9.21 are load-deflection and load-moment predictions for the Houston test for cycles 1 and 100. The proposed method is seen to be slightly conservative in the prediction of deflection, maximum moment, and depth to maximum moment for cycle 1 conditions, and somewhat more conservative for cycle 100 conditions. Presented in Fig. 9.22 are comparisons of predicted and measured behavior under cyclic loading, normalized by the cycle 1 response. Also shown in that figure are predictions using the Bogard-Matlock procedure discussed in Chapter 8. It should be recalled that cyclic response predictions with that procedure were close to measured values, although static response was quite conservative. Concern was expressed that the use of static response predictions as a basis for cyclic response predictions could lead to unconservative design for cyclic loading. This unconservatism in

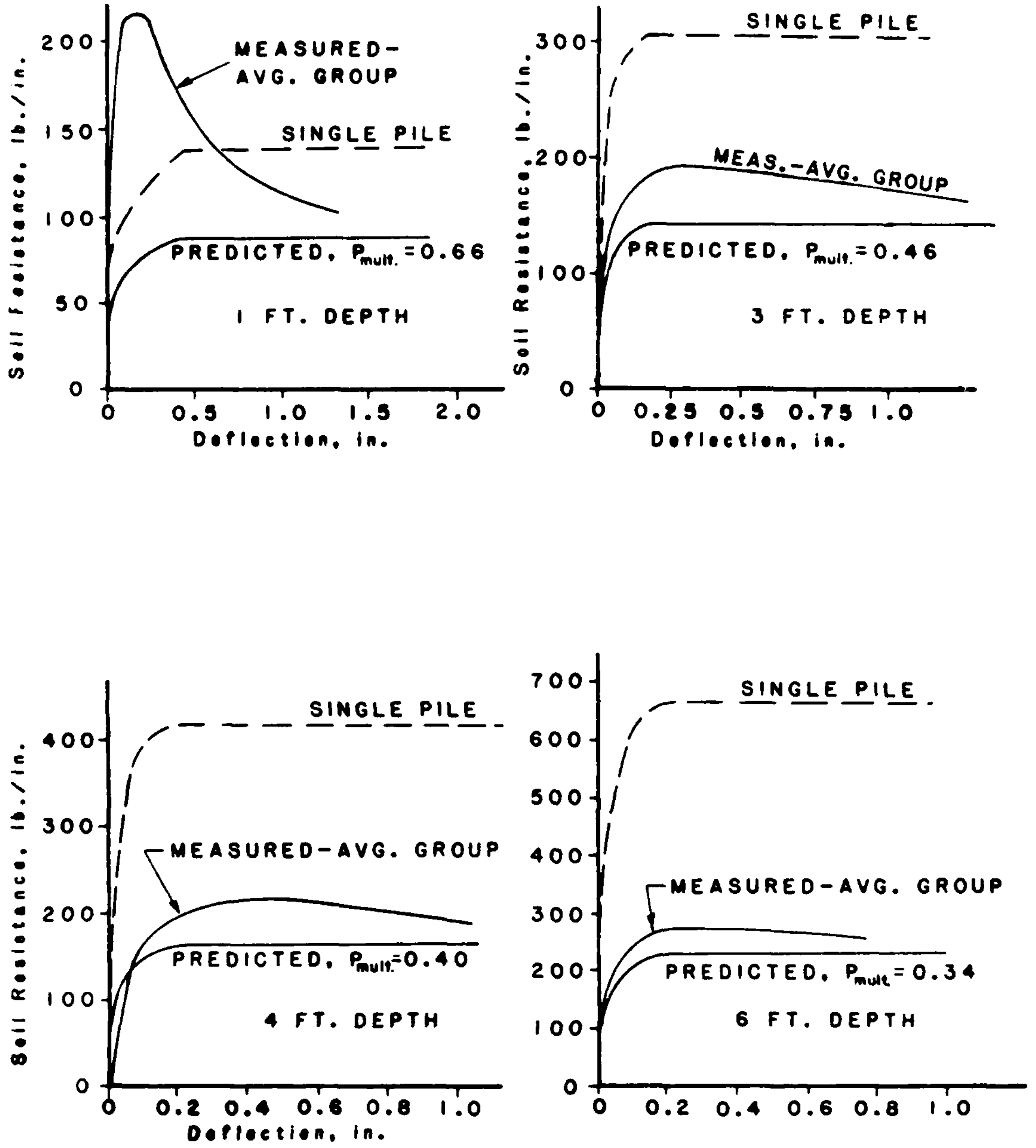

Fig. 9.14 Predicted Static P-Y Curves for Group by Proposed Method, Houston Site

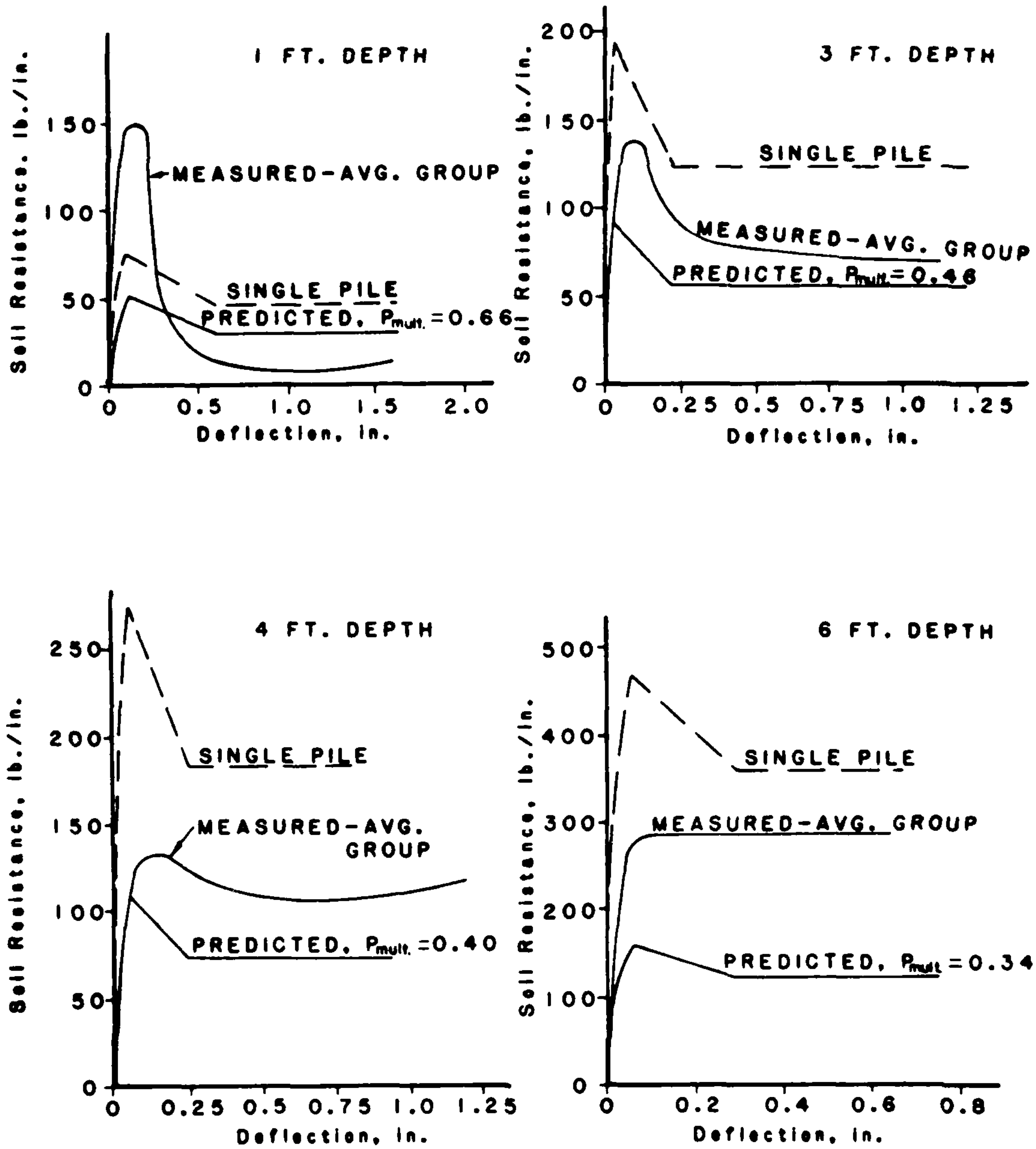

Fig. 9.15 Predicted 100 Cycle P-Y Curves for Group by Proposed Method, Houston Site

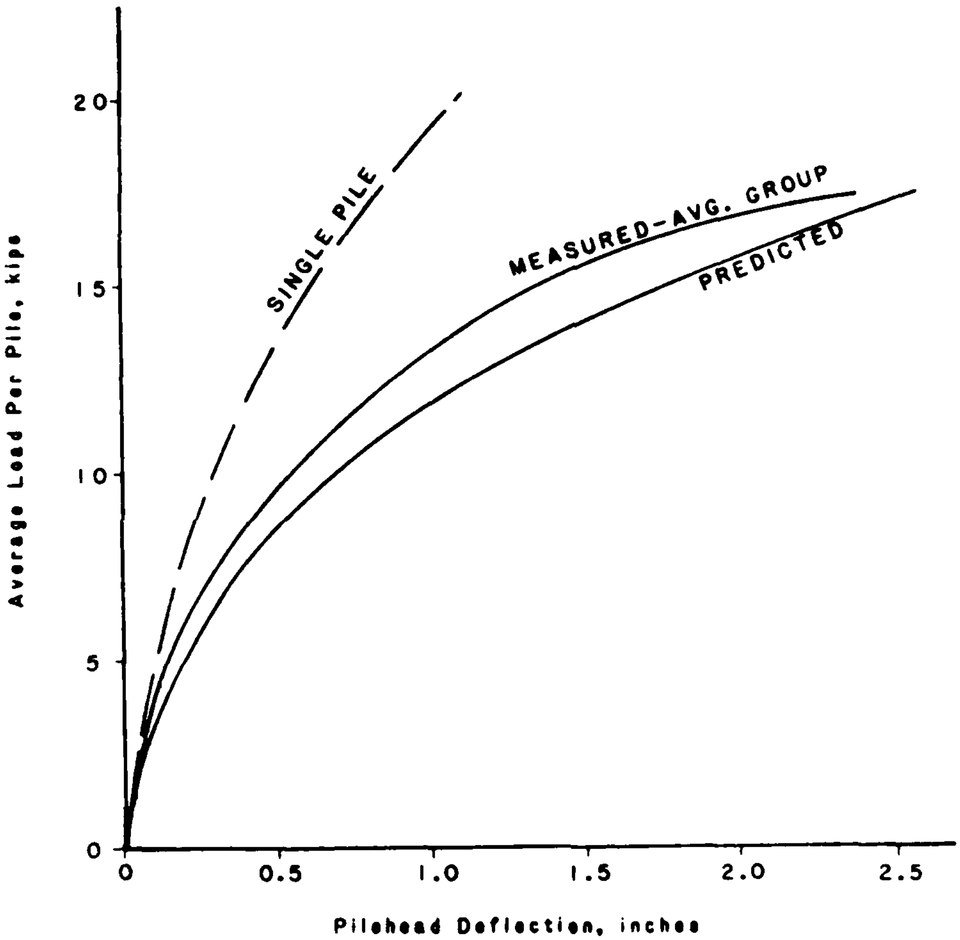

Fig. 9.16 Predicted Lateral Load vs. Deflection, for the Houston Site by the Proposed Method, Cycle 1 (Static)

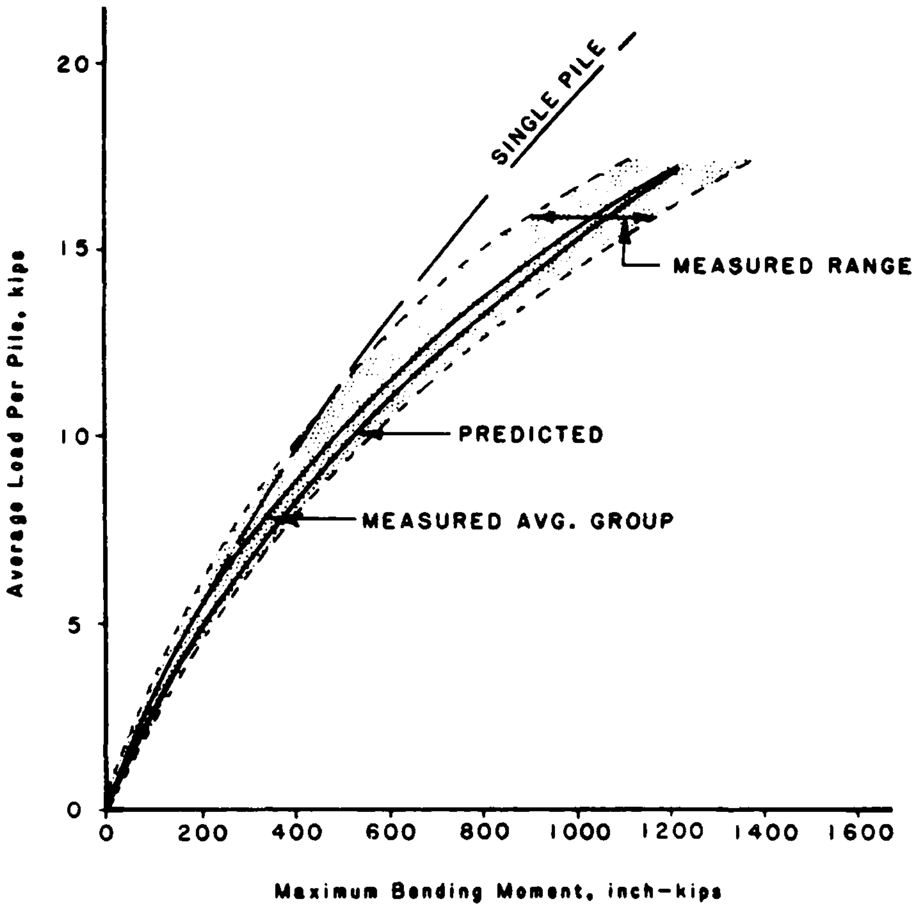

Fig. 9.17 Predicted Load vs. Maximum Moment for the Houston Site by the Proposed Method, Cycle 1 (Static)

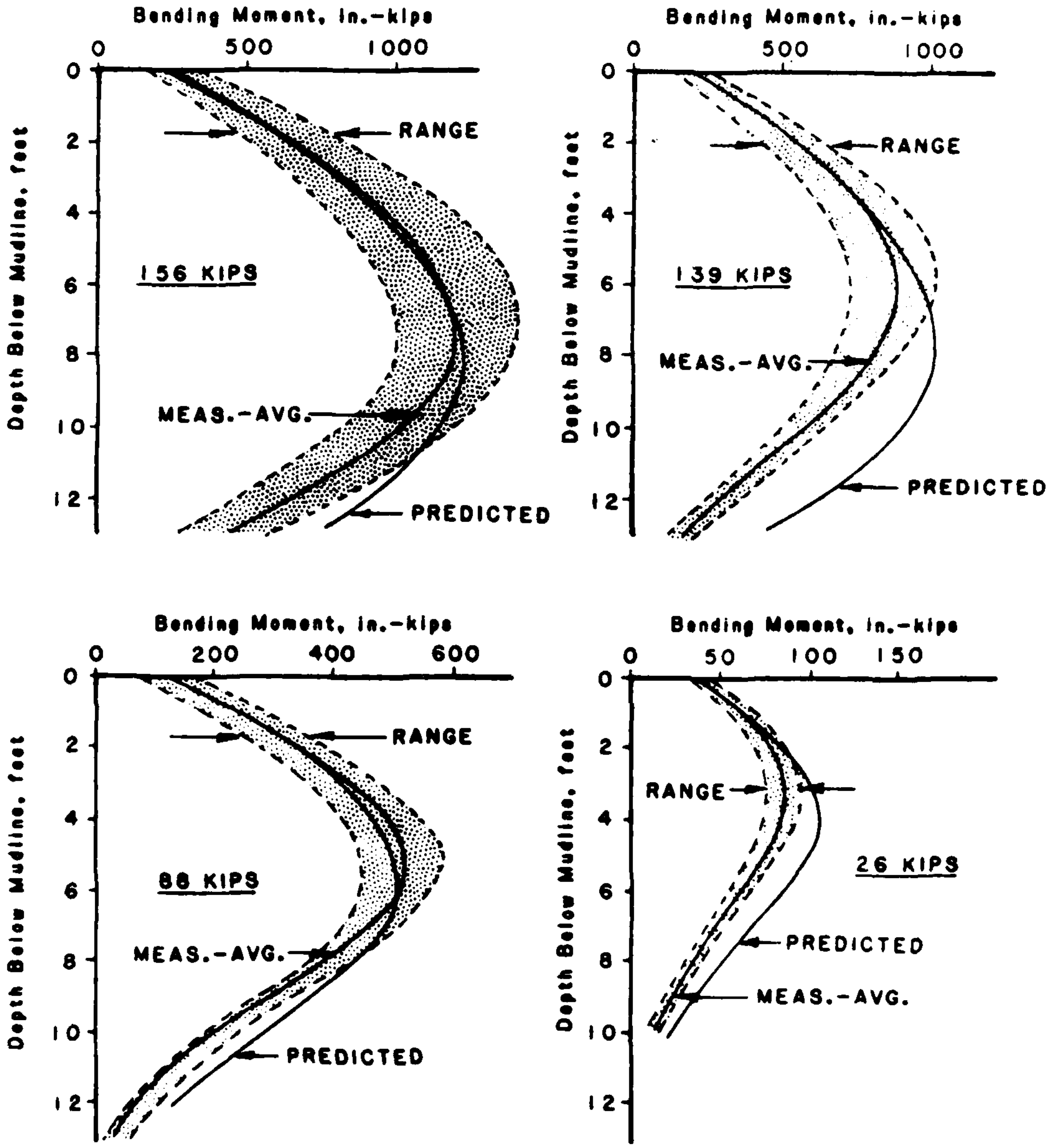

Fig. 9.18 Predicted Bending Moment vs. Depth for the Houston Site by the Proposed Method, Cycle 1 (Static)

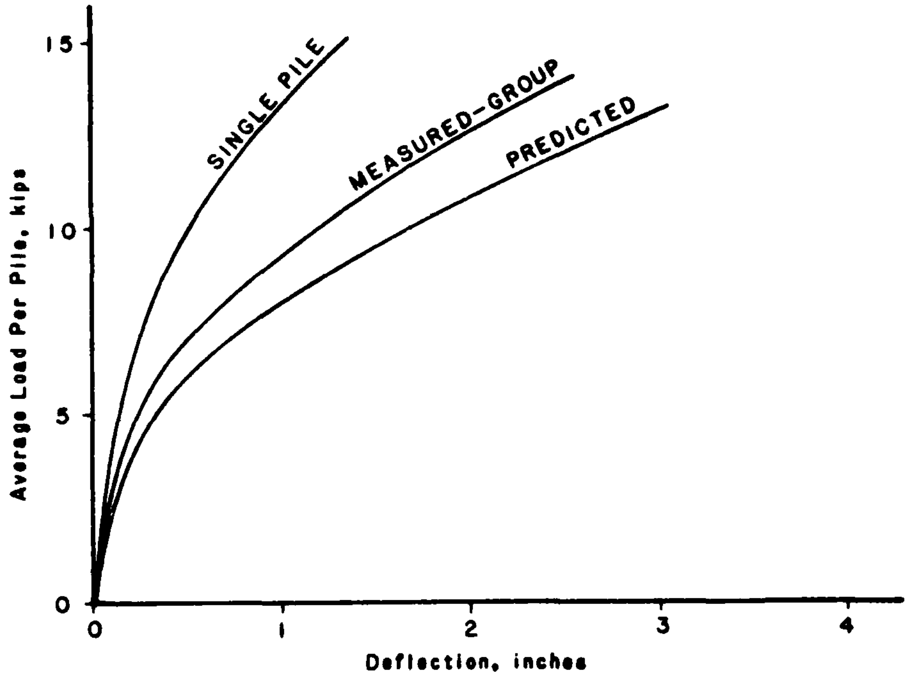

Fig. 9.19 Predicted Load vs. Deflection for the Houston Site by the Proposed Method, Cycle 100

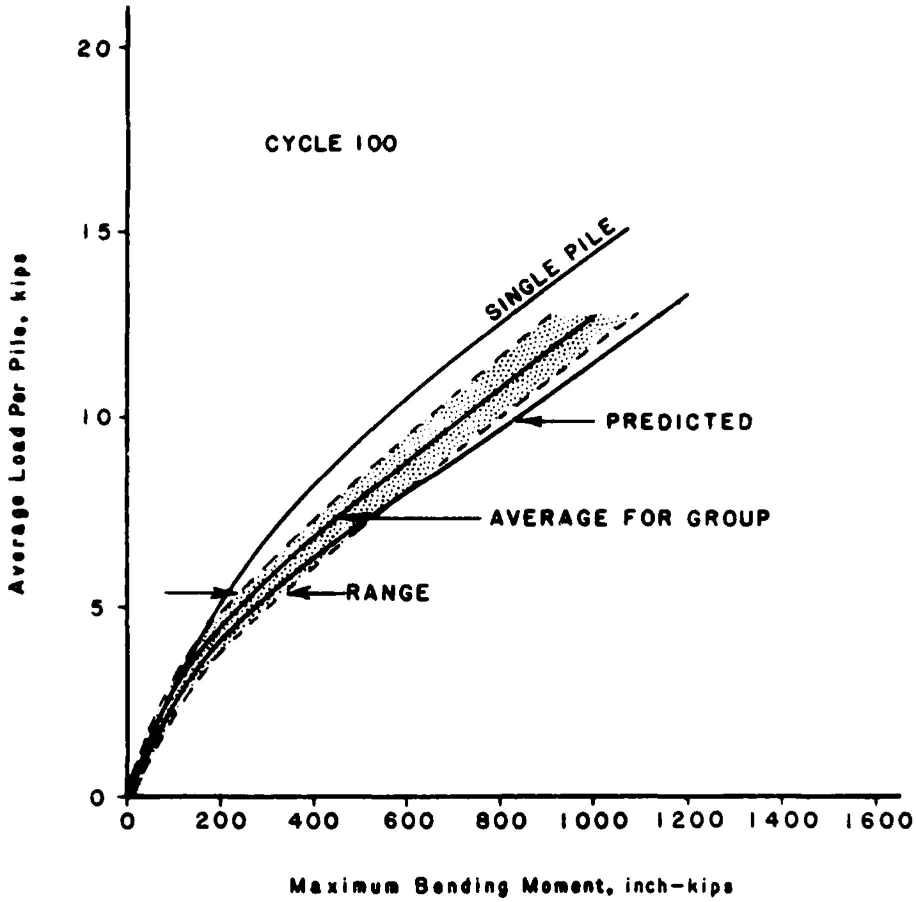

Fig. 9.20 Predicted Load vs. Maximum Moment for the Houston Site by the Proposed Method, Cycle 100

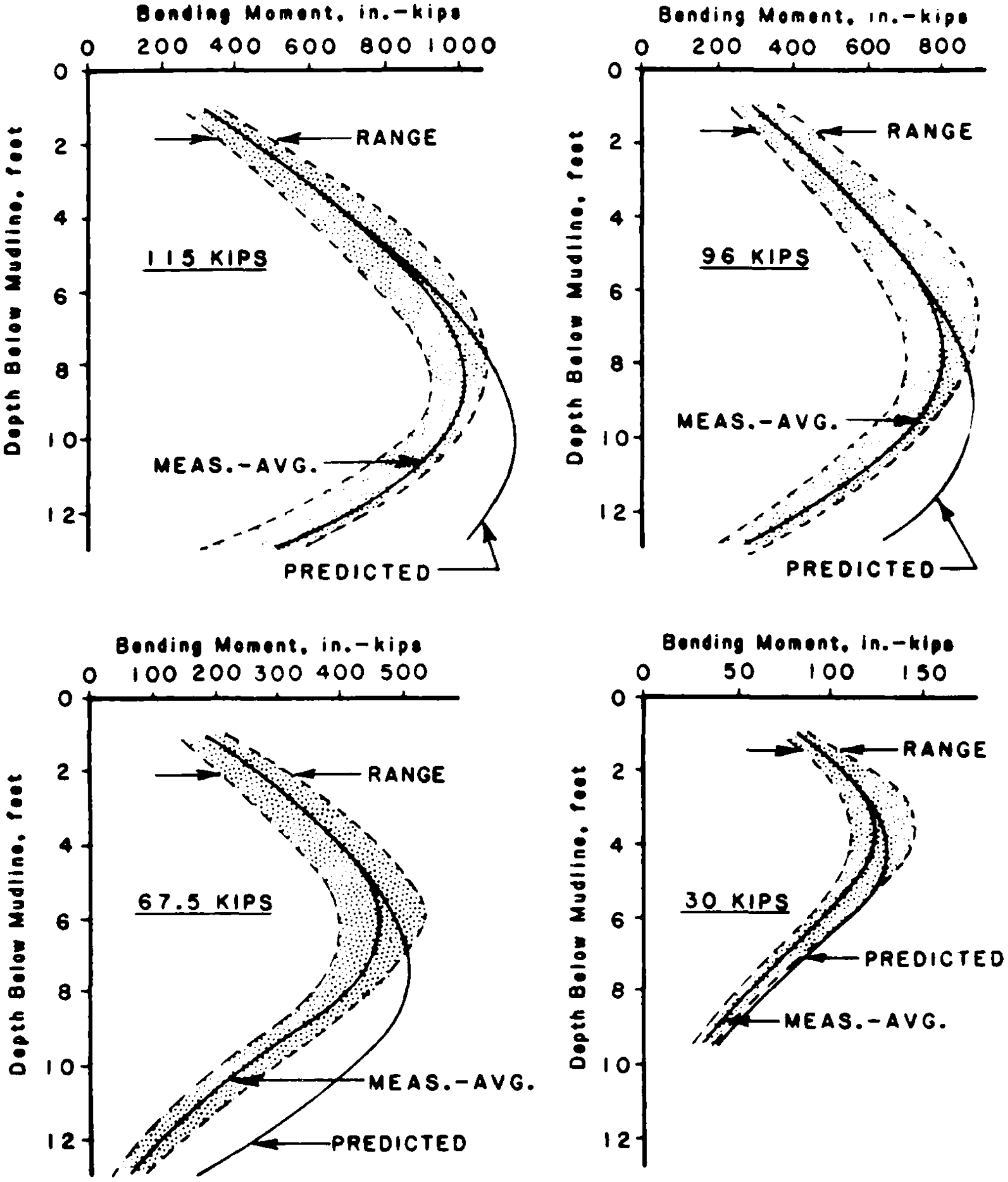

Fig. 9.21 Predicted Bending Moment vs. Depth for the Houston Site by the Proposed Method, Cycle 100

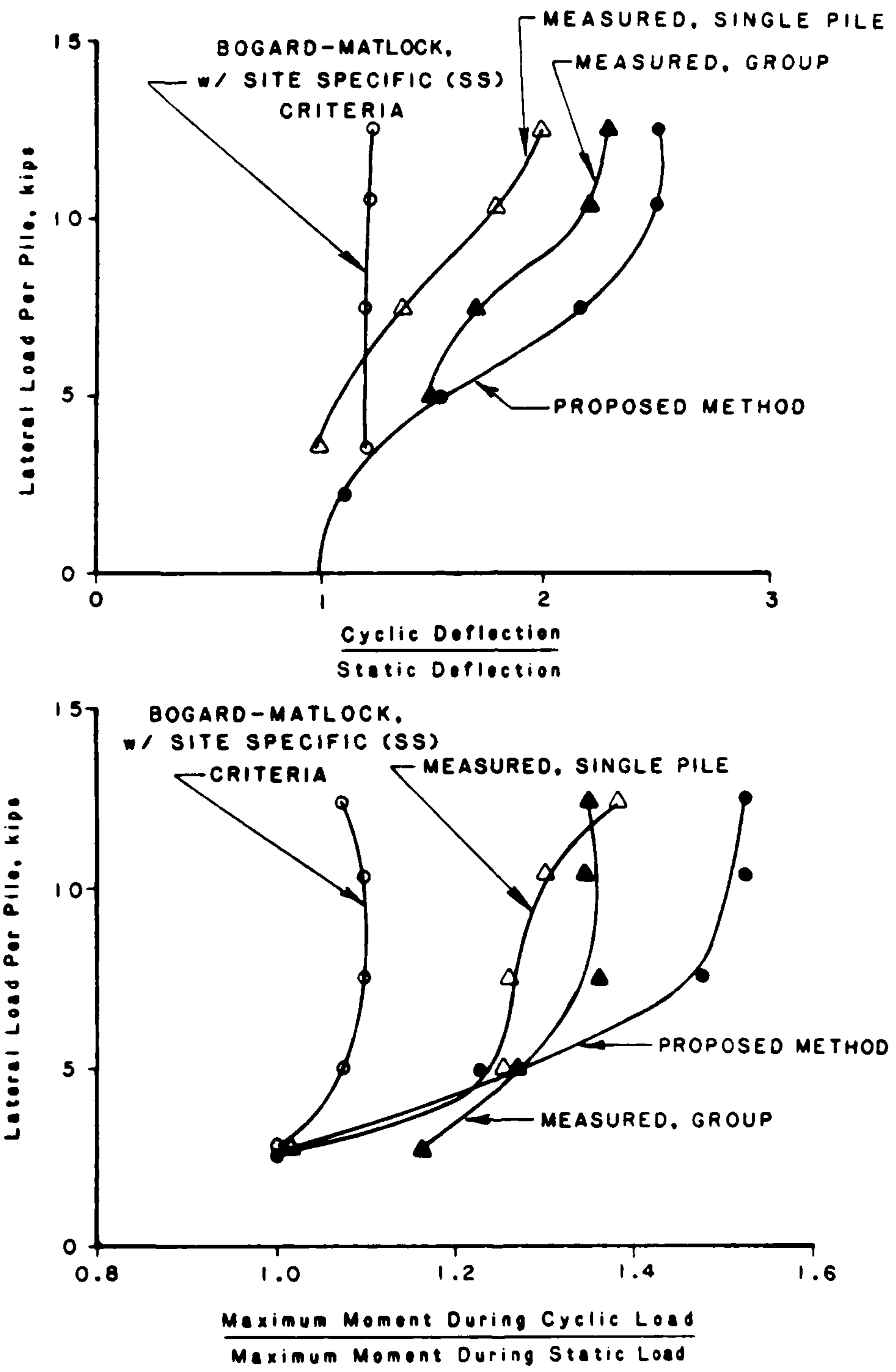

Fig. 9.22 Predicted and Measured Cyclic Response, Normalized to Static Response

cyclic response prediction was due largely to the use of a residual soil resistance equal to that of the single pile. Figure 9.22 illustrates the static to cyclic comparison and reveals the ratios of cyclic to static deflections and maximum moments to be very similar for the single pile and the average group pile. The proposed method overpredicts these ratios slightly, while the Bogard-Matlock procedure underpredicts the ratios.

In summary, it appears that for the Houston site, the proposed design method is somewhat conservative with respect to measured response of parameters important to design. This conservatism does not appear excessive, considering the uncertainties regarding the many factors that appear to affect the results. The method is only very slightly conservative for static loading, but more so for cyclic loading; much greater uncertainty is involved in predicting cyclic response, so some conservatism is perhaps desirable. No prediction is made regarding distribution of loads and stresses to the piles in the group as distinguished from the average pile; an increase of about 20% might be suggested for maximum bending moment in any pile for groups having a fairly regular pile spacing. Prudent engineering judgement on the part of the designer is certainly needed. It is important to note that the errors in design for a similar situation would be expected to be larger because of additional error in predicting the response of a single pile. The ultimate soil resistance in the example given was "calibrated" to the single-pile-test results to separate group effects from other uncertainties.

One might expect the proposed method to predict group response for the Houston site since the measurements of the Houston test were used as the basis for examining existing methods. The following paragraphs present the results of analyses of the test at Harvey, Louisiana, a test with different loading conditions and quite different soil.

HARVEY TEST

The Harvey, Louisiana experiment conducted by Matlock et al (1980) has been briefly described in Chapter 5. This experiment consisted of static and cyclic-lateral-load tests on groups of 1, 5, and 10 piles. The piles were 6.625 inches in diameter and installed in a circular arrangement in soft clay. In their analyses of the test results, Bogard and Matlock (1983) established a fit to the single pile results using a modification of current API (1979) recommendations for p-y curves in soft clay.

The proposed design method has been used to analyze the Harvey test results using two different criteria for computing the ultimate soil resistance, p_u. The first consists of the relationship proposed by Bogard and Matlock (1983) in their analysis and called SO in this discussion. The second uses the relationship of the Unified Method as described by Reese (1984). In the first method,

$$p_u = \left(3 + \frac{\bar{\sigma}_v}{S_u} + J\,\frac{x}{b}\right)cb \qquad (9.4)$$

where J is an empirical constant taken as equal to 0.25 for this site and other terms are as defined previously. The second criterion, called ST in subsequent discussion, uses Eq. 9.1. Both procedures

limit p_u to a maximum value as defined by Eq. 9.3. The variation in soil resistance as a function of deflection (the p-y curve) was computed using the API recommendations for either case of computing p_u.

Load vs. deflection predictions for the static tests at the Harvey site are presented in Figs. 9.23 and 9.24. The proposed design method is seen to provide a much better prediction of group behavior using the ST criteria for p_u, despite the fact that either the ST or SO procedure yields reasonable predictions for the single pile. Response of the group piles predicted using a large imaginary pile are quite sensitive to the distribution of p_u with depth.

Bending moment vs. depth predictions for static-load conditions are presented in Fig. 9.25. Again, better predictions are obtained for the proposed method using the ST criteria. Bending-moment predictions are somewhat less sensitive to variations in soil resistance than are deflections, and reasonable predictions are obtained using either the SO or ST procedure.

For cyclic loading, Bogard and Matlock use a peak cyclic soil resistance of 0.34 times the ultimate static soil resistance, rather than the value of 0.72 suggested in the API recommendations (1979) and by Matlock (1970). The modification of existing procedures for constructing p-y curves was made to produce agreement with the results of the single-pile test using cyclic loading. The same modification was used in constructing p-y curves for cyclic loading in this analysis.

Shown in Figs. 9.26 through 9.28 are load-deflection and moment vs. depth predictions for cyclic loading at the Harvey test site. The proposed design procedure is seen to overpredict deflection

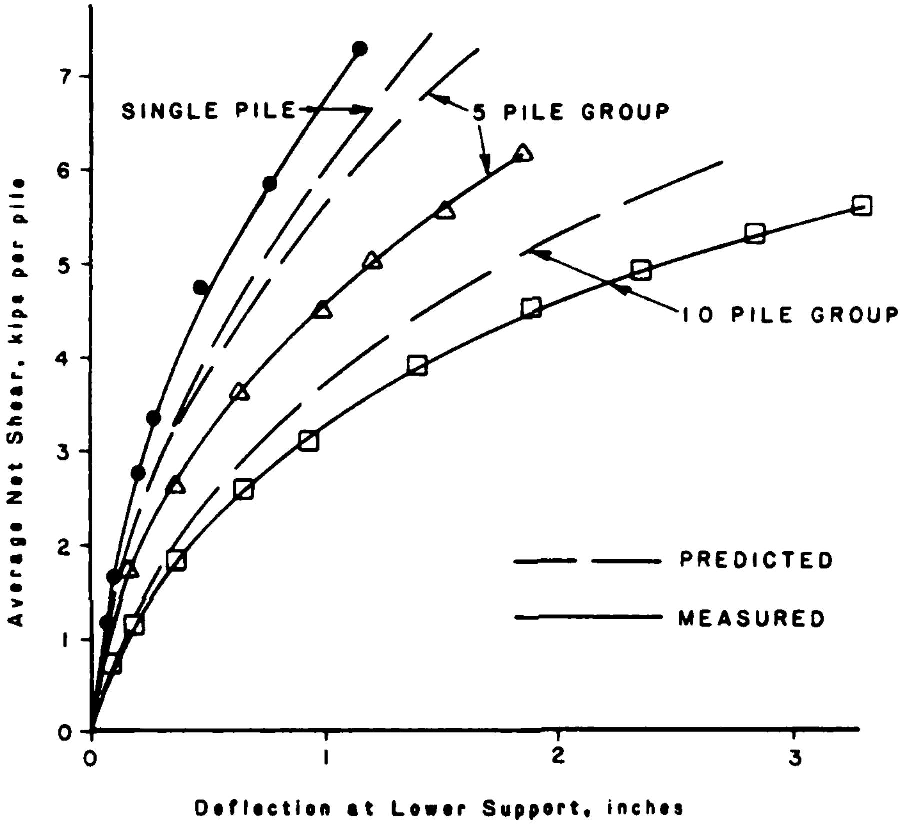

Fig. 9.23 SO Predicted vs. Measured Load-Deflection Curves, Harvey Test, Static

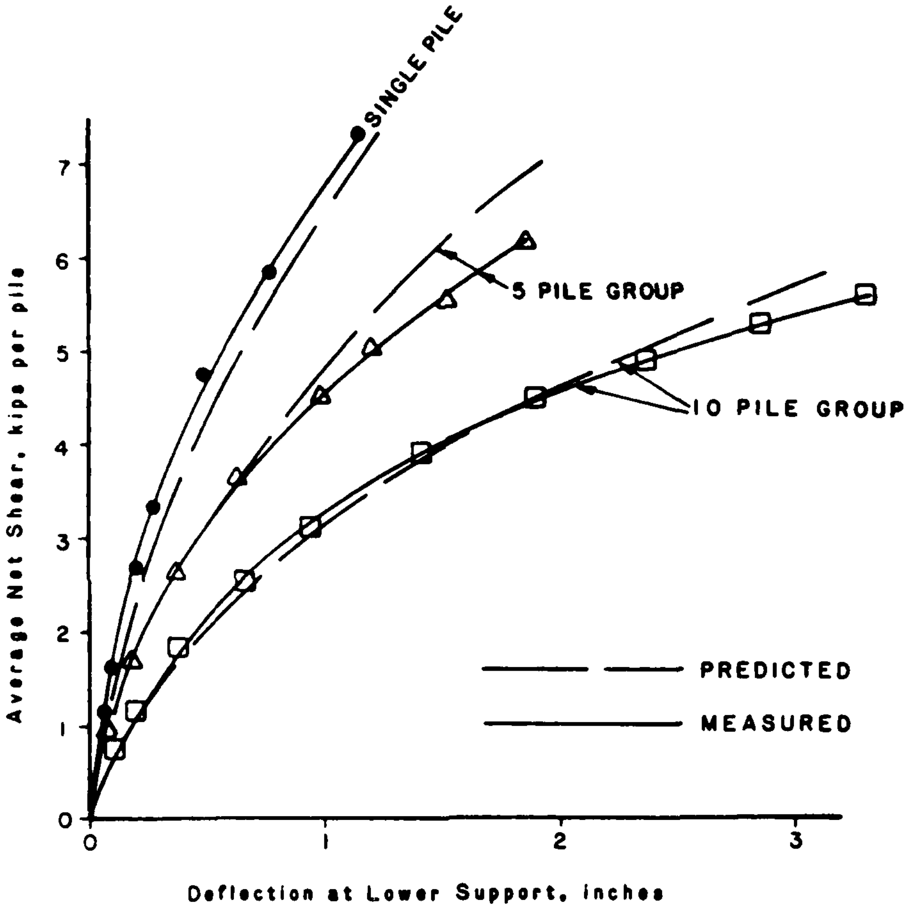

Fig. 9.24 ST Predicted vs. Measured Load-Deflection Curves, Harvey Test, Static

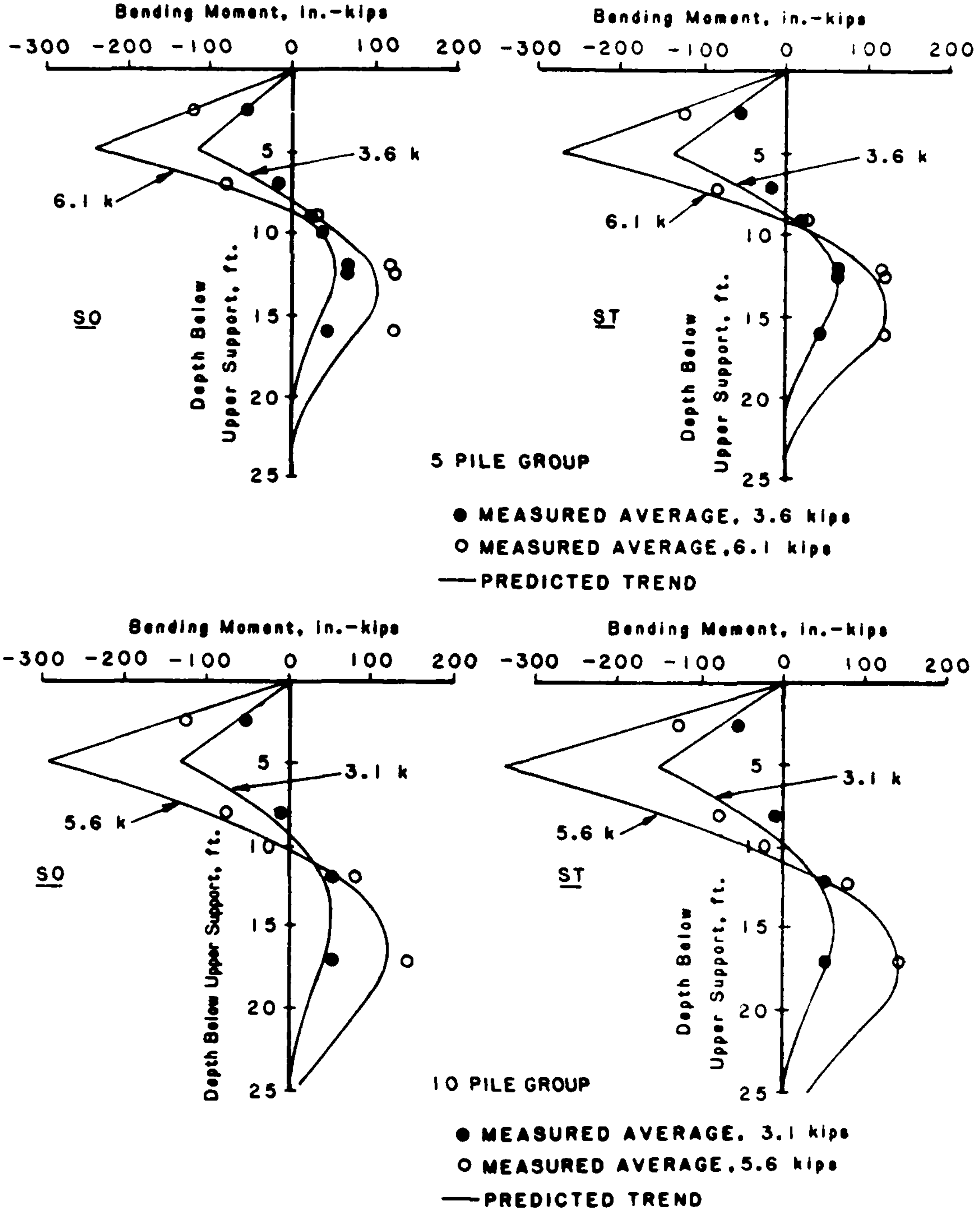

Fig. 9.25 Predicted Moment vs. Depth, Harvey Test, Static

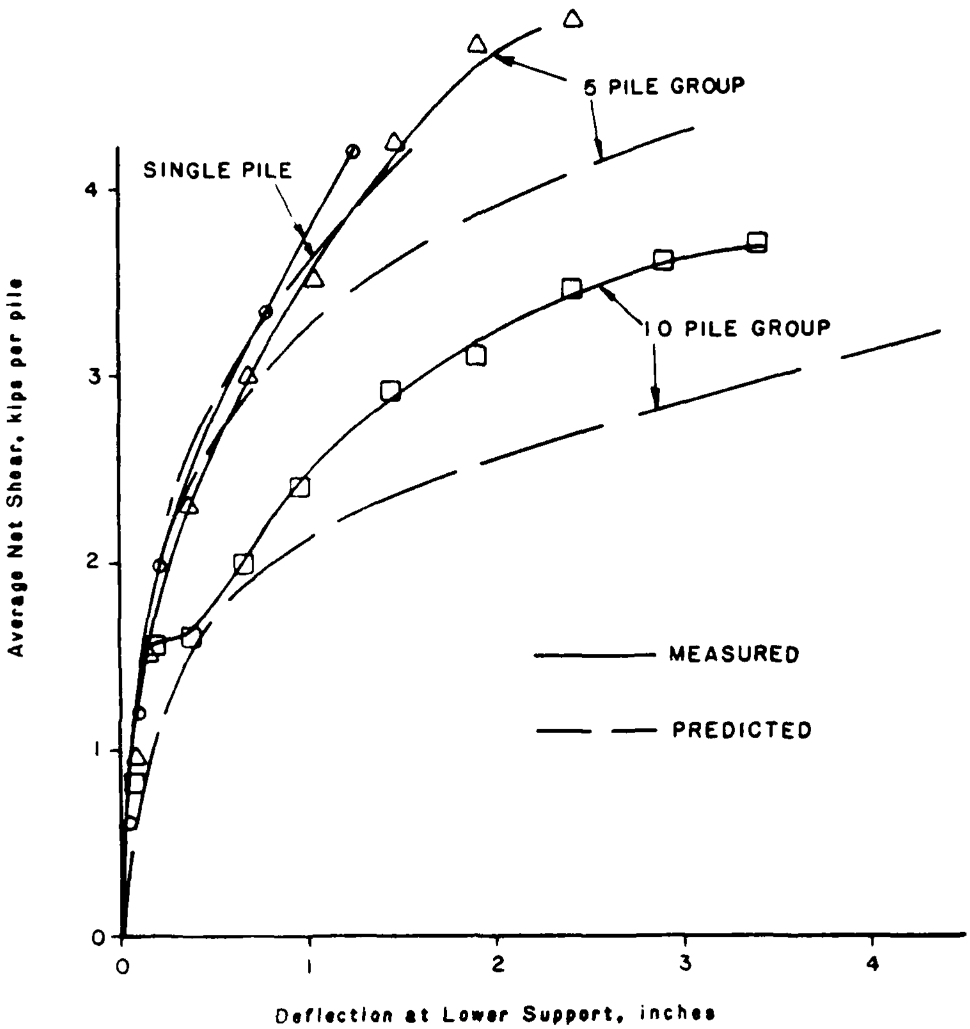

Fig. 9.26 SO Predicted vs. Measured Load-Deflection Curves, Harvey Test, Cyclic

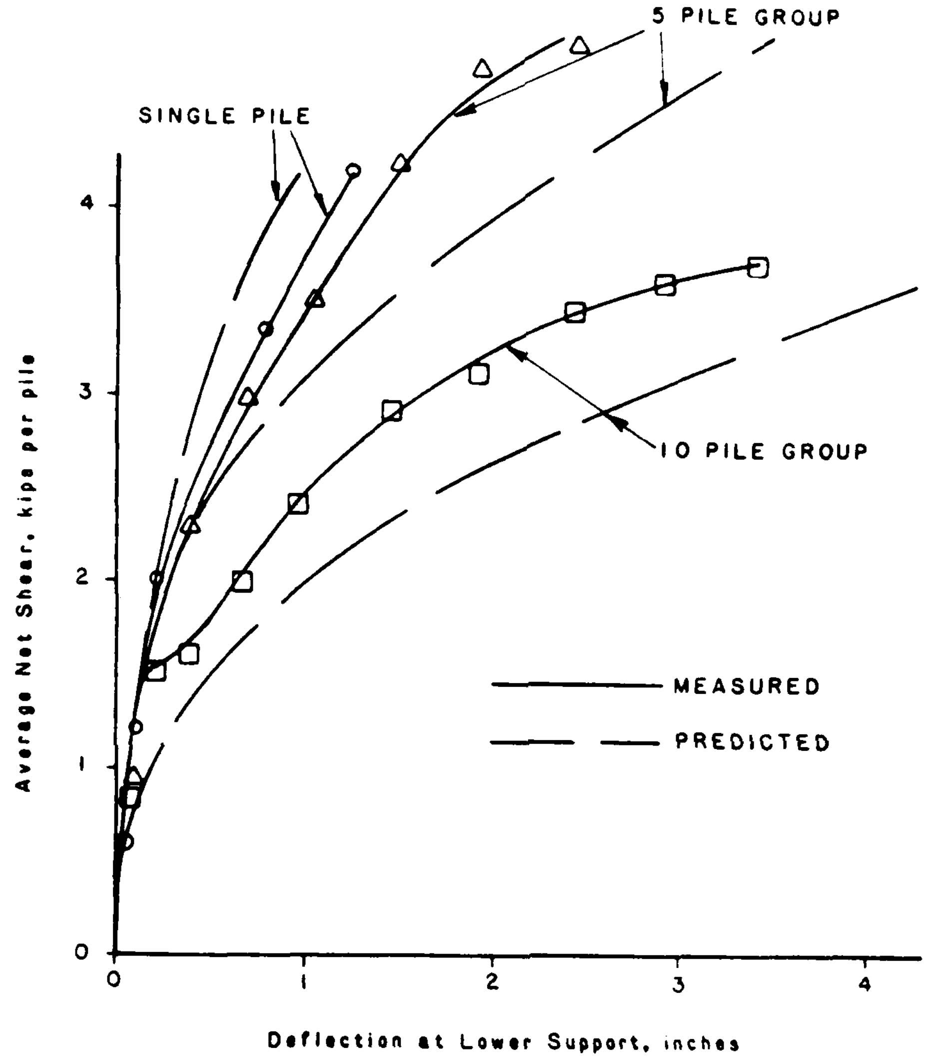

Fig. 9.27 ST Predicted vs. Measured Load-Deflection Curves, Harvey Test, Cyclic

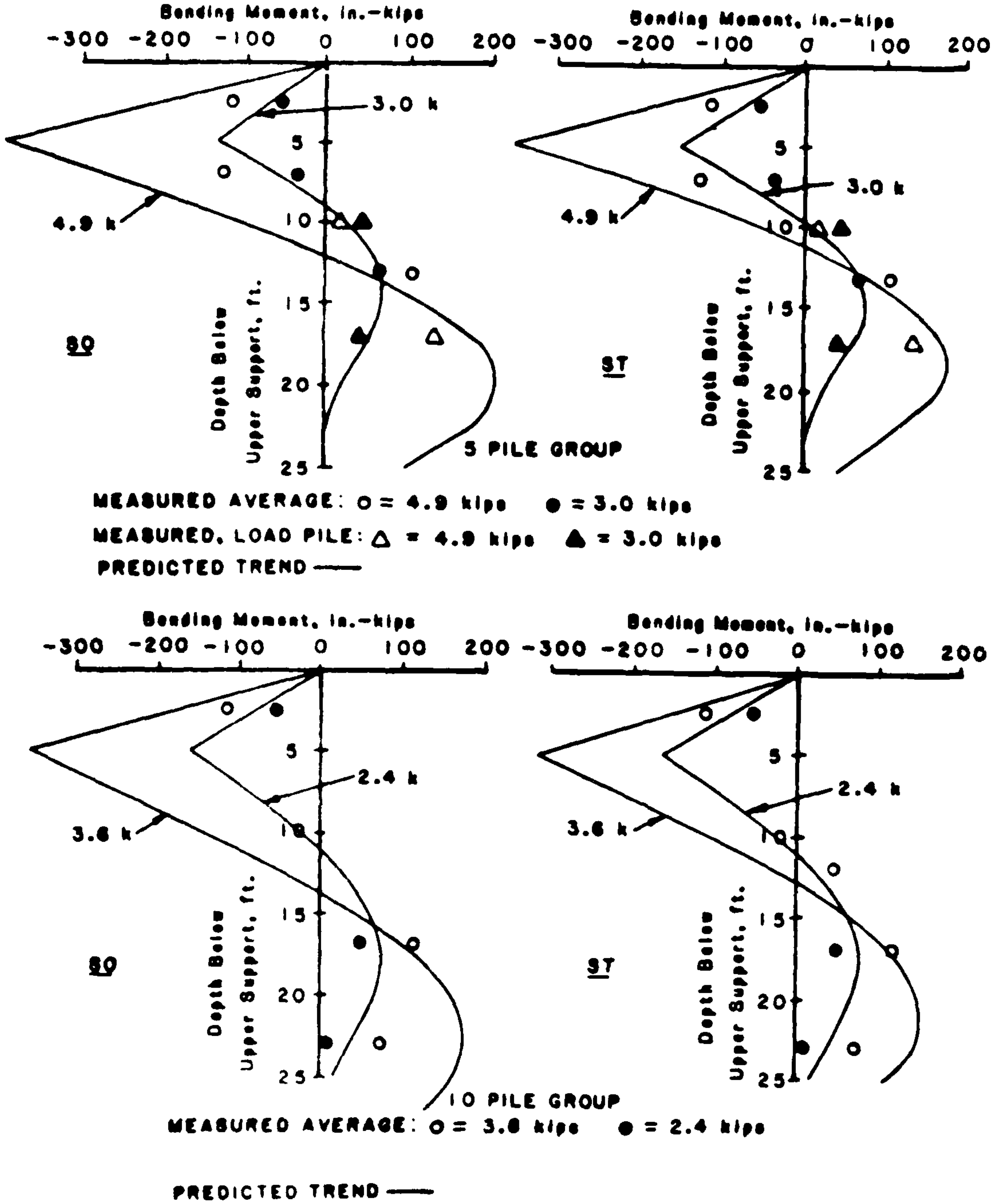

Fig. 9.28 Predicted Moment vs. Depth, Harvey Test, Cyclic

and maximum moments for the groups under cyclic loading for both the SO and ST criteria. Because the predictions of static response were relatively good, it must be concluded that the proposed method over-predicts the effect of cyclic loading on soil response for the conditions of the Harvey test site, but generally no worse so than the SO method.

An examination of the differences between the Harvey site and the Houston site might provide some insight.

1. The soils at the Harvey site had much lower undrained shear strengths than the soils at the Houston site. They were probably also more sensitive, suggesting that soil properties at the Harvey site were altered more by pile insertion.
2. The piles at the Harvey site had significant rotational restraint at the loading points, whereas the piles at the Houston site did not.
3. The combination of factors (1) and (2) would tend to make the group response at the Harvey site less sensitive to the near-surface soils, relative to the Houston test conditions. Load transfer and maximum positive bending moments are deeper and less likely influenced by surface effects with decreasing soil stiffness and pilehead rotation.
4. Drainage during cyclic loading may be a more significant factor in a stiff fissured clay like that in Houston, in that substantially more drainage may occur around a single

pile during cyclic loading than around a relatively much larger group. A relatively larger pore pressure buildup during cyclic loading might thus occur for a group of piles relative to a single pile, resulting in a relatively lower shear strength. Differences in drainage between single and group piles in the soft clay at the Harvey test site might be less significant relative to those differences at the Houston test.

Bogard and Matlock observed that the behavior of the 5-pile group during cyclic loading was very similar to that of 5 individual piles, with group effects being relatively small. The observed group effects during cyclic loading of the 10-pile group were attributed to formation of a single large gap around the entire group.

SUMMARY

A procedure for use in analyzing pile groups subjected to lateral loading has been proposed. When used with the p-y criteria described in this report, the procedure was seen to provide reasonable agreement for the group in stiff clay and for static loading of the group in soft clay. The proposed method was somewhat conservative for cyclic loading of the group in soft clay. Owing to the large uncertainty in predicting soil response to single piles subjected to lateral loading, the proposed method for pile groups emphasizes simplicity and utilizes existing design methodology to a great extent. This procedure is not offered as a rigorous analytical model, but rather as an interim procedure correlated with experimental data which

should provide generally conservative estimates of pile group behavior until a better understanding of the problem is achieved.

CHAPTER 10

SUMMARY

This chapter concludes the report with a brief summary of the findings of the investigation. The report began with a review of important concepts currently used in the design of piles and pile groups for lateral loading, followed by a description of the major experiment performed as a part of this study. The report concludes with an evaluation of existing design methods in light of the experimental evidence that was obtained.

Most design procedures that are widely used to make predictions of the response of single piles to lateral loading were seen to be based upon correlations of undrained-shear strength with soil resistance using a Winkler soil model and the results of several major field experiments. Although the procedures are well established, these methods contain a considerable degree of empiricism and are based upon relatively limited data, particularly in the case of cyclic loading.

Installation of piles may have a significant influence upon soil behavior; because soil resistance is correlated to soil properties measured prior to pile installation, such an influence is accounted for only indirectly in the empirical correlations. No such correlations exist for pile groups, and the influence of pile instal-

lation on the response of the soil and piles in a group is thought to be more significant.

A variety of models for analyzing groups of piles were reviewed. Some of these models attempt to account for pile-soil-pile interaction by elastic interaction factors; these factors are based on the assumption that the soil acts as an elastic medium and the presence of the piles in the soil is ignored. These assumptions allow computation of the effect at a given pile location from the load transfer at other nearby pile locations. Other models attempt to account for the influence of overlapping zones of shear in the soil around closely spaced piles by modification of the load-transfer curves. Maximum soil resistance is taken as that of a large "imaginary" pile which encompasses the pile group. Confidence in any of these methods is limited due to the difficulty in properly accounting for cyclic loading and due to the relative dearth of experimental evidence to support the methods.

A major experiment was performed in an attempt to provide some experimental evidence against which to test the methods. A group of 9 steel-pipe piles that were in place in stiff clay in Houston, Texas was subjected to cyclic, horizontal loading. The piles were thoroughly instrumented to measure the load in the individual piles as well as to measure the distribution of bending moment with depth in each of the piles. Comparisons with a similarly loaded single pile were made. The more significant results of the load test can be summarized as follows.

1. The deflection under load of the piles in the group is significantly greater than that of a single pile under a load equal to the average load per pile.
2. The bending moments in the piles in the group are larger and the depth to maximum moments are shifted deeper than for a single pile.
3. The maximum soil resistance for the piles in the group is greatly reduced compared to that for the single pile, and this reduction is more significant with increasing depth.
4. The soil resistance after cyclic loading at constant pile-head deflection was significantly reduced for the group piles relative to the single pile.
5. The greatest portion of the load on the group was distributed to the piles in the front row, with less load to middle row, and least to the back row. Variation of load in the piles in the group was generally 20% or less about the average, and the variation of maximum bending moments was somewhat less than 20%.

Comparison of the measured results with predictions from the various analytical models indicated that the pile-soil-pile interaction was highly nonlinear; this behavior was not predicted by elasticity-based approaches. Elasticity-based methods significantly underpredicted the deflections, moments, and depths to maximum moment at loads exceeding small values. The distribution of loads to the piles did not resemble that predicted by any elasticity-based approach. The method proposed by Bogard and Matlock (which is not

based on elasticity concepts), when used with load-transfer functions adjusted to match the single-pile results, came closest to predicting the average group response. The key factor in the Bogard-Matlock approach was the fact that a rational means of reducing the ultimate soil resistance for group effects was provided. The method is not general in that it offers no means of considering the effects of differing pile-group geometry. The method is an attempt to predict the response of an "average" pile in the group. It is also extremely sensitive to the relationship of ultimate soil resistance with depth. The Bogard-Matlock procedure seemed to predict the group behavior during cyclic loading fairly well, but the agreement was seen to be due in large part to the overprediction of static deflections and moments. When normalized to the static response, the Bogard-Matlock procedure underpredicted cyclic deflections and moments. It appears that some means of accounting for loss of soil resistance to values less than that of the single-pile values is needed.

In considering the problem of design of pile groups for lateral loading, several points are important.

1. Considerable scatter in measured values of undrained shear strength often exist, allowing for a broad range of interpretation.
2. A wide variety of correlations between undrained shear strength and maximum load transfer under static loading have been proposed.
3. The range of predictions of soil resistance for cyclic loading is even greater than the range of predictions

between undrained shear strength and maximum load transfer for static loading.

4. The three factors above make prediction of single-pile response so uncertain that complicated analytical procedures for predicting group response do not seem warranted.

A design method is proposed which consists of a relatively simple computation to produce a "p-multiplier," or a reduction factor for soil resistance. The multiplier varies with depth and is used to modify load transfer curves for both static and cyclic loading. The procedure is shown to be slightly conservative in predicting the response of the Houston piles and somewhat more so in predicting the response of the Harvey piles (Matlock et al, 1980). The procedure is not offered as a rigorous analytical model, but rather as an interim procedure for use until a better understanding and/or more experimental data are achieved.

The preceding paragraph provides an indication of research needs. More experimental evidence is needed to provide a basis for further analytical study. The ability to predict the response of pile groups to lateral loading is also hindered by the inability to characterize a site and to predict the response of single piles to lateral loading. Undrained-triaxial and vane-shear tests are typically used to predict soil response; improvement in these predictions is needed. The effects of pile installation on soil properties and on the state of stress in the soil around the piles is a subject of research interest for axially loaded piles and could be useful for the problem of laterally loaded piles as well. The behavior of piles due to cyclic

lateral loading is a combination of several complex phenomena; further studies into the behavior of cyclically-loaded piles are suggested.

APPENDICES

APPENDIX A

PROPOSED DESIGN METHOD FOR GROUPS OF PILES IN CLAY SUBJECTED TO LATERAL LOADING

The following method is proposed for the analysis of groups of regularly-spaced piles of approximately equal diameter and stiffness subjected to lateral loadings in clay soils.

1. Compute a set of p-y curves for a single pile using judgement as to the best available p-y criteria for the site. In the absence of experimental results for the site in question, the "Unified Method" (as described by Reese, 1984) is recommended.

2. Using the same p-y criteria from step 1, compute a limiting soil resistance for the average pile in the group as follows:

   a) Compute the maximum soil resistance (p_{ui}) of a large imaginary pile. For square or circular pile group configurations, a large imaginary pile having a circumference equal to the perimeter of the group is suggested. For other arrangements (such as rectangular configurations) judgement must be exercised.

   b) Compute the limiting soil resistance for an average pile in the group using the relationship:

$$p_{ug} = \frac{1}{n} p_{ui}$$

where: p_{ug} = ultimate soil resistance of the average pile in the group,

p_{ui} = ultimate soil resistance of the large imaginary pile, and

n = number of piles in the group.

3. Compute a p-multiplier, p_{mult}, by:

$$p_{mult} = p_{ug}/p_{usp}, \qquad p_{mult} \leq 1.0$$

where: p_{usp} = ultimate soil resistance of a single pile, computed in step 1.

Note that the term p_R which is used in the Unified Method does not enter into calculations of p_{mult}; p_{mult} is based entirely upon computations of p_u for the single and large imaginary piles.

4. Adjust the single-pile p-y curves for group effects by multiplying each value of soil resistance by p_{mult}. This procedure is used for both static and cyclic p-y curves.

5. Analyze the behavior of the group in terms of an "average pile" with the average pile p-y curves generated in step 5 and in terms of an average load per pile. Analyses are performed using a finite-difference technique such as used in the computer code COM622 (Reese, 1977).

6. Design the piles of the group to accommodate some range in stresses about those computed for the "average pile" in step 6. Experimental results from regularly spaced square and rectangular groups suggest a deviation of less than 20% about the average at loads approaching failure. Judgement should be exercised for groups which are not so evenly spaced or which have piles of unequal stiffness.

EXAMPLE

For a 3 x 3 group of 12-in. diameter steel pipe piles spaced 3 ft on-center, compute p-y curves for the average group pile at the 4 ft depth using the following:

shear strength at the 4 ft depth: $S_u = 6.0$ lb/sq in.

average shear strength to a depth of 4 ft: $S_{ua} = 5.0$ lb/sq in.

depth: $x = 4$ ft $= 48$ in.

effective overburden stress at the 4 ft depth: $\bar{\sigma}_v = 1.6$ lb/sq in.

1. p_{usp} calculated from the minimum

$$p_{usp} = \left(2 + \frac{\bar{\sigma}_v}{S_{ua}} + 0.833\,\frac{x}{b}\right) S_{ua} b = 339 \text{ lb/in.}$$

$$p_{usp} = \left(3 + 0.5\,\frac{x}{b}\right) S_u b = 360 \text{ lb/in.}$$

use $p_{usp} = 339$ lb/in.

Using $\varepsilon_{50} = 1\%$, $A = 1.0$, $F = 0.8$, generate curves shown in Fig. A.1.

2. Circumference of imaginary pile = 4 x 7 ft = 28 ft

Diameter of imaginary pile, $b_i = 28/\pi$ = 8.91 ft

= 107 in.

p_{ui} calculated from minimum of:

$$p_{ui} = \left(2 + \frac{\bar{\sigma}_v}{S_{ua}} + 0.833\,\frac{x}{b_i}\right) S_{ua} b_i = 1441 \text{ lb/in.}$$

$$p_{ui} = \left(3 + 0.5\,\frac{x}{b_i}\right) S_u b_i = 2070 \text{ lb/in.}$$

use $p_{ui} = 1441$ lb/in.

3. Limiting soil resistance for average group pile, p_{ug} from:

$$p_{ug} = \frac{1}{n} p_{ui} = \frac{1}{9} (1441) = 160 \text{ lb/in.}$$

4. p-multiplier, p_{mult} from:

$$p_{mult} = p_{ug}/p_{usp} = 160/339 = 0.47$$

5. Average group pile p-y curve computed by multiplying single-pile p-values by p_{mult} to obtain curves shown on Fig. A.1.

6. Using this curve and others generated similarly, use COM622 to compute deflections, moments, and stress for average group pile.

7. Size piles to accommodate allowable stresses equal to those computed in step 6, plus an additional 20%. If stiffness (EI) of piles is different from that assumed in step 6, repeat step 6. If the pile diameter is different from that assumed in step 1, repeat steps 1-7.

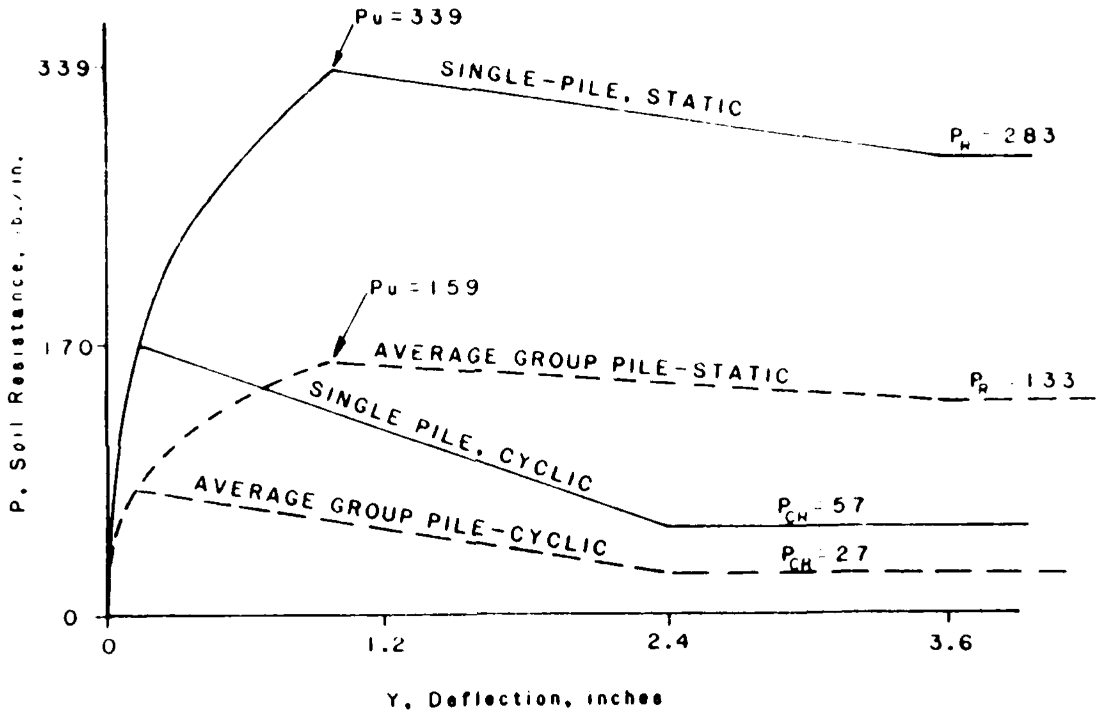

Fig A.1 Computed P-Y Curves for Example Problem

APPENDIX B

LOAD TEST DATA

****

* HOUSTON PILE GROUP STUDY *

* FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *

****

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 1
CYCLE NO. 1001
LOAD ON GROUP = 25.02 FROM BIG LOAD CELL, KIPS
= 24.94 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .08404 INCHES
WEST CAP DEFL = .07728 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	54.	65.	64.	74.	68.	54.	72.	62.	57.	63.
24.	70.	95.	83.	87.	78.	73.	84.	84.	69.	80.
36.	75.	91.	95.	90.	84.	79.	81.	95.	81.	86.
48.	75.	90.	93.	85.	87.	75.	95.	86.	77.	85.
60.	66.	80.	82.	75.	77.	70.	77.	73.	72.	75.
72.	57.	65.	67.	60.	66.	60.	62.	59.	68.	63.
84.	42.	48.	50.	44.	50.	45.	48.	43.	50.	47.
96.	31.	33.	33.	32.	39.	32.	33.	29.	34.	33.
114.	15.	15.	15.	15.	23.	17.	16.	14.	17.	16.
132.	6.	5.	6.	5.	10.	6.	8.	6.	7.	7.
156.	2.	3.	3.	2.	4.	3.	3.	2.	3.	2.
AT LOAD POINT:										
DEFL	.06	.06	.06	.06	.06	.06	.06	.05	.07	.06
SLOPE	-.00106	-.00117	-.00111	-.00119	-.00111	-.00104	-.00118	-.00117	-.00126	-.00113
LOAD	2.55	3.04	2.72	2.83	2.78	2.63	3.05	2.44	2.85	2.77

```
*************************************************************
*                 HOUSTON PILE GROUP STUDY                  *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984        *
*************************************************************
```

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 1
CYCLE NO. 2001
LOAD ON GROUP = -26.75 FROM BIG LOAD CELL, KIPS
= -24.45 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.06516 INCHES
WEST CAP DEFL = -.05649 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-57.	-59.	-67.	-62.	-70.	-61.	-63.	-62.	-56.	-63.
24.	-77.	-87.	-90.	-75.	-81.	-73.	-75.	-84.	-72.	-79.
36.	-81.	-89.	-105.	-80.	-84.	-75.	-76.	-97.	-83.	-85.
48.	-84.	-83.	-100.	-77.	-86.	-79.	-92.	-91.	-75.	-85.
60.	-71.	-71.	-88.	-73.	-74.	-71.	-77.	-79.	-64.	-74.
72.	-61.	-56.	-67.	-57.	-62.	-62.	-60.	-65.	-56.	-61.
84.	-43.	-41.	-47.	-41.	-45.	-45.	-48.	-47.	-40.	-44.
96.	-29.	-28.	-30.	-28.	-34.	-32.	-32.	-31.	-26.	-30.
114.	-12.	-12.	-11.	-12.	-17.	-13.	-15.	-13.	-17.	-13.
132.	-3.	-7.	-1.	-4.	-6.	-5.	-5.	-4.	-3.	-4.
156.	-2.	-3.	2.	0.	0.	1.	-0.	0.	1.	-0.
AT LOAD POINT:										
DEFL	-.07	-.06	-.07	-.06	-.07	-.07	-.07	-.07	-.08	-.07
SLOPE	.00118	.00126	.00111	.00098	.00108	.00107	.00108	.00114	.00131	.00113
LOAD	-2.59	-2.86	-2.63	-2.68	-2.94	-2.53	-2.77	-2.36	-2.90	-2.72

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*              HOUSTON PILE GROUP STUDY             *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 1
CYCLE NO. 1[illegible]5
LOAD ON GROUP = 26.49 FROM BIG LOAD CELL, KIPS
= 27.49 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .08756 INCHES
WEST CAP DEFL = .08122 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	63.	7[illegible].	73.	79.	69.	53.	78.	65.	58.	63.
24.	80.	94.	94.	97.	8[illegible].	82.	92.	90.	71.	87.
36.	84.	99.	106.	100.	87.	8[illegible].	89.	103.	84.	93.
48.	83.	96.	102.	96.	93.	82.	105.	94.	81.	92.
60.	72.	93.	89.	84.	85.	74.	85.	80.	77.	81.
72.	62.	55.	72.	68.	74.	63.	67.	65.	72.	67.
84.	47.	48.	54.	51.	55.	45.	53.	49.	56.	51.
96.	32.	34.	34.	34.	41.	30.	34.	30.	36.	34.
114.	14.	15.	16.	16.	24.	15.	16.	13.	18.	16.
132.	7.	4.	7.	6.	11.	5.	6.	5.	6.	6.
156.	-2.	1.	3.	3.	3.	1.	0.	2.	2.	1.
AT LOAD POINT:										
DEFL	.07	.07	.07	.06	.07	.06	.07	.07	.06	.07
SLOPE	-.00118	-.00123	-.00118	-.00132	-.00123	-.00135	-.00129	-.00134	-.00159	-.00128
LOAD	2.95	3.47	3.11	3.09	2.89	2.[illegible]5	3.43	2.67	2.92	3.05

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*              HOUSTON PILE GROUP STUDY                     *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984        *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 1
CYCLE NO. 2005
LOAD ON GROUP = -36.55 FROM BIG LOAD CELL, KIPS
= -33.40 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.11067 INCHES
WEST CAP DEFL = -.10376 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-77.	-100.	-86.	-87.	-102.	-88.	-91.	-88.	-78.	-88.
24.	-105.	-130.	-119.	-105.	-118.	-109.	-109.	-119.	-101.	-113.
36.	-112.	-135.	-142.	-115.	-126.	-115.	-113.	-138.	-120.	-124.
48.	-118.	-131.	-145.	-113.	-132.	-122.	-134.	-133.	-117.	-127.
60.	-104.	-114.	-132.	-109.	-114.	-115.	-119.	-120.	-101.	-114.
72.	-92.	-94.	-107.	-95.	-96.	-107.	-94.	-99.	-101.	-98.
84.	-67.	-77.	-78.	-72.	-72.	-79.	-77.	-73.	-68.	-73.
96.	-48.	-47.	-51.	-51.	-56.	-55.	-52.	-51.	-46.	-51.
114.	-24.	-22.	-20.	-24.	-30.	-27.	-26.	-23.	-21.	-24.
132.	-7.	-10.	-4.	-8.	-12.	-11.	-11.	-8.	-9.	-9.
156.	-3.	-4.	2.	-0.	-3.	-3.	-3.	-1.	-1.	-2.
AT LOAD POINT:										
DEFL	-.10	-.10	-.11	-.11	-.11	-.10	-.11	-.11	-.13	-.11
SLOPE	.00175	.00170	.00163	.00159	.00162	.00177	.00168	.00171	.00152	.00166
LOAD	-3.64	-4.05	-3.58	-3.50	-4.14	-3.59	-3.81	-3.26	-3.87	-3.71

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* HOUSTON PILE GROUP STUDY *

* FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *

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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 1
CYCLE NO. 1010
LOAD ON GROUP = 25.79 FROM BIG LOAD CELL, KIPS
= 26.96 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .09343 INCHES
WEST CAP DEFL = .09340 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	59.	63.	75.	76.	69.	60.	77.	67.	61.	68.
24.	78.	97.	97.	95.	84.	87.	93.	95.	74.	88.
36.	88.	95.	111.	102.	92.	90.	94.	110.	88.	97.
48.	90.	97.	110.	100.	99.	88.	111.	102.	85.	98.
60.	81.	88.	98.	92.	90.	78.	92.	88.	81.	88.
72.	71.	73.	79.	75.	79.	62.	74.	72.	73.	73.
84.	54.	56.	60.	56.	59.	47.	57.	53.	59.	55.
96.	38.	40.	40.	39.	45.	32.	38.	34.	39.	38.
114.	18.	19.	19.	19.	25.	15.	19.	15.	20.	19.
132.	9.	5.	9.	8.	12.	5.	8.	6.	9.	8.
156.	-1.	0.	5.	4.	3.	1.	2.	2.	3.	2.
AT LOAD POINT:										
DEFL	.08	.08	.07	.06	.07	.07	.08	.07	.07	.07
SLOPE	-.00130	-.00132	-.00133	-.00139	-.00133	-.00133	-.00135	-.00143	-.00163	-.00135
LOAD	2.69	3.09	3.11	3.02	2.90	3.10	3.37	2.68	3.00	3.00

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*                HOUSTON PILE GROUP STUDY                   *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984        *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 1
CYCLE NO. 2010
LOAD ON GROUP = -33.34 FROM BIG LOAD CELL, KIPS
= -30.34 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.10515 INCHES
WEST CAP DEFL = -.10001 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-67.	-93.	-78.	-80.	-95.	-81.	-83.	-81.	-71.	-81.
24.	-92.	-122.	-111.	-96.	-111.	-101.	-100.	-112.	-92.	-104.
36.	-100.	-126.	-133.	-107.	-117.	-107.	-104.	-129.	-112.	-115.
48.	-108.	-121.	-134.	-105.	-123.	-114.	-123.	-123.	-109.	-113.
60.	-96.	-104.	-121.	-101.	-106.	-107.	-110.	-111.	-93.	-105.
72.	-86.	-84.	-98.	-88.	-69.	-100.	-87.	-91.	-95.	-91.
84.	-63.	-62.	-71.	-66.	-55.	-74.	-70.	-65.	-62.	-67.
96.	-45.	-41.	-46.	-46.	-50.	-54.	-46.	-46.	-42.	-46.
114.	-22.	-18.	-17.	-21.	-27.	-24.	-23.	-21.	-19.	-21.
132.	-6.	-9.	-3.	-7.	-11.	-11.	-10.	-7.	-8.	-8.
156.	-3.	-3.	2.	-3.	-3.	-2.	-3.	-1.	-1.	-1.
AT LOAD POINT:										
DEFL	-.10	-.09	-.10	-.10	-.10	-.10	-.10	-.10	-.12	-.10
SLOPE	.00149	.00144	.00146	.00137	.00151	.00161	.00152	.00157	.00133	.00145
LOAD	-3.21	-3.63	-3.19	-3.15	-3.85	-3.26	-3.43	-3.03	-3.38	-3.37

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*                HOUSTON PILE GROUP STUDY                   *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984        *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 1
CYCLE NO. 1020
LOAD ON GROUP = 27.45 FROM BIG LOAD CELL, KIPS
= 28.22 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .09579 INCHES
WEST CAP DEFL = .08533 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	61.	67.	78.	74.	71.	62.	79.	72.	62.	70.
24.	81.	94.	104.	96.	87.	91.	97.	100.	78.	92.
36.	93.	112.	121.	106.	97.	95.	101.	117.	93.	103.
48.	97.	115.	120.	108.	104.	93.	118.	110.	91.	105.
60.	88.	96.	109.	98.	96.	83.	99.	96.	87.	95.
72.	78.	77.	87.	81.	86.	66.	85.	79.	78.	79.
84.	61.	60.	66.	62.	64.	50.	63.	59.	65.	61.
96.	41.	42.	44.	41.	49.	34.	40.	37.	41.	41.
114.	19.	22.	20.	19.	27.	16.	19.	16.	22.	20.
132.	9.	7.	9.	7.	13.	5.	7.	7.	7.	8.
156.	-2.	1.	2.	3.	3.	0.	1.	1.	1.	1.
AT LOAD POINT:										
DEFL	.08	.08	.09	.07	.08	.07	.08	.08	.08	.08
SLOPE	-.00140	-.00141	-.00137	-.00144	-.00137	-.00119	-.00144	-.00153	-.00179	-.00144
LOAD	2.77	3.24	3.28	3.10	3.07	3.31	3.52	2.83	3.10	3.14

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*                 HOUSTON PILE GROUP STUDY                   *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984         *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 1
CYCLE NO. 2020
LOAD ON GROUP = -33.28 FROM BIG LOAD CELL, KIPS
= -30.28 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.10765 INCHES
WEST CAP DEFL = -.09798 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-67.	-94.	-79.	-81.	-97.	-82.	-76.	-80.	-71.	-81.
24.	-93.	-123.	-112.	-98.	-112.	-107.	-93.	-111.	-92.	-104.
36.	-106.	-126.	-132.	-108.	-117.	[illegible]	-101.	-128.	-112.	-114.
48.	-107.	-119.	-133.	-106.	-122.	-111.	-119.	-122.	-110.	-117.
60.	-95.	-101.	-119.	-101.	-105.	-103.	-112.	-110.	-93.	-104.
72.	-85.	-91.	-96.	-86.	-87.	-99.	-89.	-90.	-96.	-90.
84.	-62.	-59.	-69.	-63.	-66.	-73.	-73.	-66.	-62.	-66.
96.	-45.	-39.	-45.	-45.	-50.	-54.	-51.	-45.	-42.	-45.
114.	-23.	-17.	-17.	-21.	-27.	[illegible]	-24.	-21.	-19.	-22.
132.	-6.	-8.	-3.	-7.	-11.	-13.	-11.	-7.	-9.	-8.
156.	-3.	-2.	1.	-0.	-3.	-3.	-3.	-1.	-2.	-2.
AT LOAD POINT:										
DEFL	-.09	-.08	-.10	-.10	-.10	-.09	-.10	-.10	-.11	-.10
SLOPE	.00163	.00156	.00148	.00147	.00144	.00138	.00146	.00156	.00135	.00151
LOAD	-3.26	-3.06	-3.28	-3.17	-3.92	-3.22	-3.15	-2.95	-3.35	-3.35

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*              HOUSTON PILE GROUP STUDY             *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 1
CYCLE NO. 1050
LOAD ON GROUP = 28.62 FROM BIG LOAD CELL, KIPS
= 30.22 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .10290 INCHES
WEST CAP DEFL = .06785 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	63.	74.	82.	74.	75.	65.	78.	77.	66.	73.
24.	87.	105.	113.	97.	92.	97.	100.	107.	83.	98.
36.	102.	115.	133.	110.	103.	101.	108.	126.	101.	111.
48.	106.	116.	134.	114.	112.	99.	127.	121.	99.	114.
60.	98.	106.	122.	107.	103.	89.	110.	106.	94.	104.
72.	88.	86.	99.	90.	92.	73.	89.	89.	86.	88.
84.	68.	65.	76.	68.	69.	55.	69.	67.	70.	67.
96.	48.	46.	51.	48.	54.	37.	46.	44.	46.	47.
114.	23.	23.	25.	22.	30.	13.	22.	19.	23.	23.
132.	12.	7.	10.	8.	13.	5.	8.	7.	8.	9.
156.	-0.	1.	3.	3.	2.	-0.	0.	1.	2.	1.
AT LOAD POINT:										
DEFL	.09	.09	.10	.08	.09	.08	.09	.10	.09	.09
SLOPE	-.00152	-.00153	-.00158	-.00153	-.00151	-.00128	-.00158	-.00171	-.00197	-.00158
LOAD	2.90	3.56	3.49	3.26	3.37	3.57	3.69	3.08	3.30	3.35

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*                  HOUSTON PILE GROUP STUDY                    *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984           *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 1
CYCLE NO. 2050
LOAD ON GROUP = -32.23 FROM BIG LOAD CELL, KIPS
= -28.49 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.10020 INCHES
WEST CAP DEFL = -.09809 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-63.	-91.	-78.	-77.	-93.	-77.	-67.	-75.	-63.	-76.
24.	-87.	-116.	-107.	-93.	-106.	-94.	-82.	-103.	-85.	-97.
36.	-92.	-117.	-126.	-102.	-110.	-97.	-91.	-119.	-106.	-107.
48.	-98.	-109.	-121.	-100.	-113.	-101.	-108.	-112.	-104.	-107.
60.	-86.	-91.	-106.	-93.	-96.	-95.	-103.	-100.	-87.	-95.
72.	-76.	-73.	-84.	-78.	-79.	-89.	-82.	-81.	-90.	-81.
84.	-54.	-53.	-59.	-56.	-59.	-64.	-68.	-59.	-56.	-59.
96.	-38.	-32.	-37.	-39.	-45.	-48.	-47.	-40.	-37.	-40.
114.	-18.	-14.	-13.	-18.	-24.	-21.	-24.	-18.	-17.	-19.
132.	-3.	-6.	-2.	-6.	-11.	-4.	-11.	-6.	-7.	-7.
156.	-2.	-2.	2.	-1.	-3.	-3.	-4.	-1.	-1.	-2.
AT LOAD POINT:										
DEFL	-.08	-.07	-.08	-.09	-.09	-.08	-.09	-.09	-.11	-.09
SLOPE	.00150	.00135	.00143	.00140	.00136	.00148	.00136	.00141	.00120	.00139
LOAD	-3.12	-3.57	-3.19	-2.97	-3.76	-3.13	-2.66	-2.74	-3.06	-3.17

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* HOUSTON PILE GROUP STUDY *

* FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *

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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 1
CYCLE NO. 1100
LOAD ON GROUP = 29.82 FROM BIG LOAD CELL, KIPS
= 31.20 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .10504 INCHES
WEST CAP DEFL = .08899 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	68.	80.	89.	78.	82.	71.	81.	82.	72.	73.
24.	94.	115.	124.	103.	100.	105.	105.	115.	92.	106.
36.	111.	125.	147.	118.	111.	111.	116.	136.	111.	121.
48.	115.	126.	149.	123.	121.	110.	136.	131.	109.	125.
60.	107.	114.	136.	118.	112.	99.	121.	116.	104.	114.
72.	97.	93.	112.	100.	100.	82.	99.	98.	96.	97.
84.	76.	71.	87.	77.	77.	63.	78.	75.	77.	76.
96.	55.	50.	60.	55.	60.	44.	53.	50.	52.	53.
114.	29.	24.	30.	27.	35.	23.	26.	23.	28.	27.
132.	16.	7.	13.	11.	15.	9.	11.	9.	11.	11.
156.	3.	0.	6.	4.	3.	2.	1.	2.	4.	3.
AT LOAD POINT:										
DEFL	.10	.10	.11	.09	.09	.09	.10	.10	.10	.10
SLOPE	-.00159	-.00165	-.00172	-.00162	-.00161	-.00137	-.00166	-.00189	-.00212	-.00169
LOAD	2.93	3.78	3.55	3.36	3.57	3.63	3.72	3.23	3.43	3.47

HOUSTON PILE GROUP STUDY

FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984

LOAD NO. 1
CYCLE NO. 2100
LOAD ON GROUP = -30.71 FROM BIG LOAD CELL, KIPS
= -27.63 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.10947 INCHES
WEST CAP DEFL = -.10514 INCHES

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-63.	-92.	-81.	-77.	-96.	-77.	-68.	-78.	-60.	-77.
24.	-90.	-118.	-115.	-94.	-111.	-93.	-85.	-110.	-81.	-100.
36.	-97.	-122.	-135.	-103.	-115.	-98.	-95.	-131.	-105.	-111.
48.	-105.	-115.	-131.	-102.	-118.	-103.	-114.	-123.	-107.	-113.
60.	-92.	-98.	-117.	-95.	-100.	-96.	-109.	-109.	-93.	-101.
72.	-81.	-79.	-91.	-79.	-83.	-91.	-88.	-89.	-99.	-86.
84.	-58.	-58.	-63.	-56.	-61.	-65.	-73.	-64.	-62.	-62.
96.	-40.	-35.	-39.	-39.	-47.	-48.	-50.	-43.	-42.	-43.
114.	-18.	-16.	-12.	-17.	-25.	-21.	-25.	-19.	-18.	-19.
132.	-1.	-8.	1.	-4.	-10.	-9.	-10.	-6.	-7.	-6.
156.	1.	-4.	5.	1.	-3.	-1.	-3.	-1.	0.	-1.
AT LOAD POINT:										
DEFL	-.09	-.08	-.09	-.09	-.10	-.09	-.10	-.10	-.12	-.10
SLOPE	.00160	.00141	.00153	.00141	.00149	.00153	.00152	.00152	.00114	.00145
LOAD	-3.07	-3.55	-3.18	-2.90	-3.76	-3.[illegible]7	-2.81	-2.76	-2.73	-3.07

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*            HOUSTON PILE GROUP STUDY               *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 2
CYCLE NO. 1001
LOAD ON GROUP = 65.00 FROM BIG LOAD CELL, KIPS
= 66.85 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .29072 INCHES
WEST CAP DEFL = .28320 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	157.	182.	200.	192.	181.	175.	194.	168.	147.	177.
24.	212.	259.	271.	244.	223.	231.	258.	232.	197.	236.
36.	245.	293.	320.	280.	257.	253.	286.	278.	240.	272.
48.	284.	312.	335.	294.	312.	271.	327.	286.	245.	296.
60.	275.	297.	325.	295.	303.	263.	304.	280.	236.	287.
72.	254.	251.	289.	268.	276.	247.	258.	265.	280.	266.
84.	218.	213.	241.	218.	219.	203.	226.	229.	228.	222.
96.	149.	158.	180.	169.	181.	155.	162.	174.	164.	156.
114.	87.	83.	98.	92.	115.	90.	91.	96.	98.	95.
132.	46.	32.	43.	42.	59.	43.	46.	44.	48.	45.
156.	11.	5.	12.	15.	18.	14.	13.	12.	16.	13.
AT LOAD POINT:										
DEFL	.28	.29	.29	.26	.27	.26	.29	.29	.29	.28
SLOPE	-.00395	-.00400	-.00414	-.00412	-.00398	-.00357	-.00415	-.00430	-.00472	-.00410
LOAD	6.50	8.17	7.88	7.82	7.30	7.45	8.45	6.34	6.93	7.43

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*                  HOUSTON PILE GROUP STUDY                  *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984         *
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		NORTH	
	D	G	A
	F	E	H
	B	I	C
	PILE LEGEND		

LOAD NO. 2
CYCLE NO. 2001
LOAD ON GROUP = -67.45 FROM BIG LOAD CELL, KIPS
= -62.62 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.31007 INCHES
WEST CAP DEFL = -.31353 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-147.	-192.	-182.	-172.	-199.	-155.	-153.	-177.	-171.	-172.
24.	-200.	-259.	-255.	-218.	-236.	-218.	-188.	-245.	-217.	-226.
36.	-222.	-288.	-308.	-248.	-267.	-253.	-235.	-297.	-271.	-256.
48.	-282.	-331.	-326.	-252.	-299.	-293.	-256.	-307.	-277.	-283.
60.	-251.	-288.	-320.	-267.	-300.	-284.	-275.	-303.	-278.	-285.
72.	-231.	-251.	-288.	-252.	-254.	-292.	-258.	-279.	-295.	-258.
84.	-189.	-218.	-237.	-210.	-195.	-238.	-222.	-227.	-208.	-217.
96.	-159.	-161.	-176.	-171.	-164.	-172.	-190.	-167.	-134.	-155.
114.	-92.	-91.	-88.	-96.	-98.	-90.	-107.	-82.	-73.	-91.
132.	-40.	-43.	-28.	-40.	-46.	-40.	-53.	-31.	-30.	-39.
156.	-8.	-13.	1.	-7.	-12.	-7.	-14.	-5.	-5.	-8.
AT LOAD POINT:										
DEFL	-.29	-.27	-.29	-.28	-.27	-.29	-.30	-.33	-.33	-.29
SLOPE	.00419	.00416	.00429	.00395	.00395	.00416	.00389	.00416	.00434	.00408
LOAD	-6.83	-7.43	-7.01	-6.56	-7.67	-7.35	-6.10	-6.67	-7.26	-6.95

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*              HOUSTON PILE GROUP STUDY             *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 2
CYCLE NO. 1005
LOAD ON GROUP = 54.05 FROM BIG LOAD CELL, KIPS
= 57.53 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .28967 INCHES
WEST CAP DEFL = .28209 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	134.	152.	173.	164.	159.	155.	166.	147.	123.	153.
24.	181.	223.	241.	216.	201.	213.	232.	209.	182.	211.
36.	216.	259.	296.	258.	237.	238.	272.	255.	229.	251.
48.	261.	285.	321.	282.	293.	260.	307.	275.	232.	280.
60.	262.	274.	317.	295.	286.	262.	303.	278.	232.	279.
72.	263.	245.	283.	275.	257.	223.	266.	262.	276.	262.
84.	228.	205.	236.	226.	214.	187.	233.	224.	222.	219.
96.	156.	157.	176.	174.	179.	153.	170.	171.	157.	166.
114.	91.	56.	97.	93.	116.	89.	89.	95.	96.	95.
132.	47.	34.	44.	39.	60.	44.	43.	43.	51.	45.
156.	11.	5.	15.	13.	19.	16.	11.	12.	19.	13.
AT LOAD POINT:										
DEFL	.28	.28	.28	.26	.27	.26	.28	.28	.28	.28
SLOPE	-.00355	-.00374	-.00380	-.00380	-.00374	-.00328	-.00387	-.00403	-.00456	-.00382
LOAD	5.52	6.93	6.77	6.59	6.25	6.68	7.32	5.58	5.89	6.39

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*              HOUSTON PILE GROUP STUDY           *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 2
CYCLE NO. 2005
LOAD ON GROUP = -57.13 FROM BIG LOAD CELL, KIPS
= -52.76 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.30005 INCHES
WEST CAP DEFL = -.29222 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-125.	-171.	-157.	-152.	-173.	-133.	-136.	-148.	-140.	-148.
24.	-176.	-236.	-226.	-197.	-209.	-193.	-172.	-212.	-175.	-199.
36.	-204.	-254.	-261.	-226.	-251.	-227.	-225.	-265.	-233.	-242.
48.	-275.	-278.	-302.	-240.	-277.	-258.	-259.	-283.	-249.	-271.
60.	-234.	-259.	-301.	-246.	-276.	-262.	-267.	-284.	-263.	-257.
72.	-218.	-246.	-273.	-234.	-237.	-277.	-251.	-266.	-274.	-253.
84.	-179.	-208.	-226.	-197.	-187.	-226.	-228.	-221.	-205.	-219.
96.	-150.	-153.	-171.	-160.	-156.	-163.	-174.	-165.	-137.	-159.
114.	-88.	-91.	-67.	-92.	-95.	-87.	-97.	-84.	-75.	-88.
132.	-40.	-45.	-29.	-39.	-46.	-39.	-46.	-32.	-31.	-39.
156.	-8.	-15.	3.	-6.	-12.	-6.	-12.	-5.	-5.	-7.
AT LOAD POINT:										
DEFL	-.28	-.26	-.28	-.27	-.27	-.28	-.29	-.31	-.33	-.28
SLOPE	.00393	.00383	.00397	.00366	.00381	.00390	.00385	.00399	.00366	.00384
LOAD	-5.86	-6.33	-5.92	-5.81	-5.62	-5.88	-5.28	-5.47	-5.60	-5.85

HOUSTON PILE GROUP STUDY

FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 2
CYCLE NO. 1010
LOAD ON GROUP = 55.36 FROM BIG LOAD CELL, KIPS
= 58.10 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .28419 INCHES
WEST CAP DEFL = .27314 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	136.	146.	176.	165.	160.	155.	170.	146.	124.	153.
24.	183.	213.	246.	218.	204.	217.	237.	209.	184.	212.
36.	217.	244.	301.	261.	242.	243.	277.	256.	234.	253.
48.	262.	271.	327.	287.	299.	265.	312.	277.	237.	282.
60.	264.	266.	322.	301.	304.	268.	308.	282.	236.	283.
72.	267.	248.	288.	281.	272.	223.	278.	268.	283.	268.
84.	231.	216.	240.	232.	218.	191.	237.	229.	226.	224.
96.	158.	173.	178.	190.	183.	155.	172.	175.	158.	170.
114.	94.	99.	98.	95.	118.	91.	90.	97.	98.	98.
132.	49.	43.	45.	41.	61.	46.	43.	44.	31.	47.
156.	11.	6.	15.	13.	18.	16.	12.	12.	19.	14.
AT LOAD POINT:										
DEFL	.28	.29	.29	.26	.27	.26	.28	.28	.27	.28
SLOPE	-.00362	-.00375	-.00383	-.00397	-.00363	-.00337	-.00398	-.00415	-.00483	-.00392
LOAD	5.59	6.85	6.89	6.68	5.33	6.79	7.46	5.58	5.93	6.45

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*                HOUSTON PILE GROUP STUDY                    *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984         *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 2
CYCLE NO. 2010
LOAD ON GROUP = -60.30 FROM BIG LOAD CELL, KIPS
= -56.03 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.29741 INCHES
WEST CAP DEFL = -.29560 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-129.	-171.	-164.	-158.	-160.	-140.	-140.	-155.	-145.	-154.
24.	-181.	-235.	-237.	-205.	-218.	-202.	-178.	-223.	-182.	-207.
36.	-209.	-258.	-292.	-234.	-251.	-233.	-234.	-276.	-237.	-250.
48.	-285.	-293.	-311.	-248.	-287.	-231.	-267.	-292.	-258.	-280.
60.	-240.	-275.	-308.	-250.	-273.	-271.	-275.	-292.	-272.	-273.
72.	-225.	-252.	-276.	-240.	-242.	-283.	-255.	-270.	-281.	-258.
84.	-183.	-214.	-226.	-200.	-189.	-226.	-230.	-222.	-208.	-211.
96.	-151.	-158.	-168.	-160.	-156.	-152.	-173.	-163.	-138.	-159.
114.	-86.	-91.	-83.	-89.	-93.	-84.	-95.	-81.	-73.	-86.
132.	-37.	-44.	-25.	-37.	-42.	-36.	-46.	-29.	-29.	-35.
156.	-7.	-14.	4.	-5.	-10.	-5.	-12.	-3.	-4.	-6.
AT LOAD POINT:										
DEFL	-.26	-.26	-.28	-.27	-.26	-.28	-.29	-.29	-.33	-.28
SLOPE	.00403	.00391	.00406	.00379	.00388	.00403	.00389	.00405	.00351	.00392
LOAD	-6.17	-6.50	-6.33	-6.14	-7.03	-6.43	-5.65	-5.89	-5.39	-6.23

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*              HOUSTON PILE GROUP STUDY             *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 2
CYCLE NO. 1020
LOAD ON GROUP = 53.75 FROM BIG LOAD CELL, KIPS
= 56.14 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .28387 INCHES
WEST CAP DEFL = .27422 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	130.	134.	172.	153.	158.	159.	168.	143.	121.	150.
24.	177.	198.	242.	214.	202.	217.	234.	206.	183.	208.
36.	211.	231.	298.	254.	242.	243.	274.	253.	237.	250.
48.	257.	262.	325.	281.	300.	267.	309.	276.	239.	280.
60.	264.	263.	321.	317.	323.	273.	310.	281.	239.	288.
72.	272.	252.	288.	284.	275.	229.	281.	269.	282.	270.
84.	237.	223.	242.	239.	220.	192.	242.	232.	230.	229.
96.	159.	193.	182.	188.	185.	157.	177.	180.	160.	175.
114.	96.	106.	103.	103.	121.	93.	93.	101.	100.	102.
132.	52.	46.	48.	46.	64.	48.	46.	47.	54.	50.
156.	13.	8.	16.	16.	20.	18.	13.	14.	20.	15.
AT LOAD POINT:										
DEFL	.28	.30	.30	.26	.27	.26	.28	.23	.28	.23
SLOPE	-.00366	-.00374	-.00382	-.00399	-.00388	-.00351	-.00399	-.00421	-.00486	-.00395
LOAD	5.29	6.17	6.68	6.49	6.23	6.74	7.29	5.47	5.80	6.24

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*             HOUSTON PILE GROUP STUDY              *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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LOAD NO. 2
CYCLE NO. 2020
LOAD ON GROUP = -53.67 FROM BIG LOAD CELL, KIPS
= -49.38 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.29211 INCHES
WEST CAP DEFL = -.29537 INCHES

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-118.	-154.	-146.	-144.	-165.	-123.	-126.	-139.	-129.	-138.
24.	-166.	-216.	-214.	-190.	-202.	-191.	-162.	-202.	-161.	-189.
36.	-198.	-249.	-268.	-220.	-246.	-223.	-218.	-255.	-214.	-232.
48.	-275.	-257.	-291.	-236.	-275.	-282.	-250.	-275.	-240.	-256.
60.	-229.	-253.	-291.	-239.	-244.	-255.	-260.	-277.	-258.	-257.
72.	-214.	-244.	-265.	-232.	-232.	-272.	-244.	-260.	-275.	-249.
84.	-178.	-207.	-220.	-194.	-181.	-221.	-221.	-215.	-203.	-205.
96.	-146.	-153.	-165.	-156.	-151.	-158.	-169.	-159.	-137.	-155.
114.	-85.	-92.	-84.	-89.	-92.	-83.	-94.	-81.	-74.	-86.
132.	-37.	-47.	-27.	-37.	-43.	-36.	-46.	-30.	-30.	-37.
156.	-7.	-16.	3.	-5.	-10.	-5.	-11.	-4.	-4.	-7.
AT LOAD POINT:										
DEFL	-.27	-.26	-.27	-.27	-.26	-.28	-.28	-.28	-.33	-.28
SLOPE	.00382	.00369	.00384	.00361	.00364	.00332	.00378	.00390	.00340	.00372
LOAD	-5.58	-5.67	-5.50	-5.55	-5.29	-5.61	-4.94	-5.16	-5.09	-5.49

HOUSTON PILE GROUP STUDY

FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 2
CYCLE NO. 1050
LOAD ON GROUP = 53.52 FROM BIG LOAD CELL, KIPS
= 58.98 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .29954 INCHES
WEST CAP DEFL = .27970 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	145.	143.	179.	159.	171.	192.	178.	146.	118.	163.
24.	199.	215.	251.	225.	220.	237.	252.	209.	179.	221.
36.	238.	257.	311.	270.	268.	273.	290.	263.	235.	267.
48.	285.	296.	343.	299.	331.	303.	330.	291.	267.	305.
60.	290.	296.	346.	339.	366.	308.	334.	299.	243.	314.
72.	303.	278.	315.	305.	312.	266.	306.	288.	283.	295.
84.	255.	244.	271.	261.	245.	221.	263.	251.	241.	253.
96.	175.	206.	208.	209.	206.	182.	198.	198.	173.	195.
114.	110.	110.	121.	119.	136.	136.	106.	113.	108.	114.
132.	61.	46.	59.	57.	73.	56.	55.	56.	60.	58.
156.	18.	5.	20.	21.	25.	22.	19.	18.	24.	19.
AT LOAD POINT:										
DEFL	.31	.32	.32	.30	.31	.30	.31	.31	.31	.31
SLOPE	-.00407	-.00435	-.00409	-.00414	-.00419	-.00386	-.00428	-.00422	-.00443	-.00413
LOAD	6.04	7.00	6.97	5.19	6.76	7.32	7.40	5.71	5.59	6.55

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* HOUSTON PILE GROUP STUDY *

* FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *

****

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 2
CYCLE NO. 2050
LOAD ON GROUP = -51.25 FROM BIG LOAD CELL, KIPS
= -44.72 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.27387 INCHES
WEST CAP DEFL = -.28320 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-108.	-143.	-138.	-125.	-156.	-102.	-107.	-125.	-138.	-124.
24.	-154.	-212.	-206.	-167.	-194.	-192.	-138.	-185.	-136.	-175.
36.	-187.	-245.	-262.	-198.	-240.	-222.	-199.	-238.	-186.	-220.
48.	-269.	-251.	-286.	-214.	-266.	-277.	-224.	-261.	-213.	-252.
60.	-225.	-255.	-283.	-219.	-224.	-247.	-239.	-265.	-235.	-244.
72.	-208.	-231.	-255.	-214.	-219.	-260.	-223.	-245.	-257.	-235.
84.	-186.	-193.	-208.	-178.	-159.	-213.	-201.	-199.	-184.	-191.
96.	-132.	-136.	-153.	-142.	-139.	-147.	-152.	-145.	-123.	-141.
114.	-71.	-82.	-73.	-78.	-82.	-73.	-83.	-71.	-57.	-76.
132.	-27.	-41.	-20.	-29.	-35.	-30.	-36.	-23.	-26.	-30.
156.	-1.	-17.	6.	3.	-6.	-0.	-6.	1.	-1.	-3.
AT LOAD POINT:										
DEFL	-.26	-.24	-.26	-.24	-.25	-.25	-.25	-.25	-.28	-.26
SLOPE	.00359	.00321	.00375	.00336	.00351	.00372	.00335	.00376	.00335	.00351
LOAD	-5.18	-5.01	-5.32	-4.95	-5.84	-5.19	-4.24	-4.70	-4.05	-4.97

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*              HOUSTON PILE GROUP STUDY                 *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984   *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 2
CYCLE NO. 1100
LOAD ON GROUP = 49.10 FROM BIG LOAD CELL, KIPS
= 52.09 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .31861 INCHES
WEST CAP DEFL = .28144 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	116.	127.	143.	130.	160.	185.	162.	142.	114.	144.
24.	162.	194.	200.	200.	210.	227.	232.	204.	177.	201.
36.	198.	239.	251.	244.	267.	258.	276.	259.	236.	249.
48.	249.	284.	293.	279.	337.	310.	315.	295.	283.	294.
60.	309.	293.	319.	329.	378.	314.	336.	314.	252.	315.
72.	319.	286.	315.	313.	326.	273.	325.	307.	294.	306.
84.	286.	257.	295.	276.	256.	228.	285.	272.	283.	271.
96.	227.	235.	249.	228.	217.	191.	219.	220.	184.	219.
114.	134.	125.	164.	135.	145.	113.	119.	128.	114.	131.
132.	76.	57.	86.	66.	78.	60.	62.	65.	63.	68.
156.	22.	10.	27.	23.	27.	25.	21.	22.	24.	22.
AT LOAD POINT:										
DEFL	.34	.33	.34	.31	.32	.31	.33	.33	.33	.33
SLOPE	-.00418	-.00453	-.00418	-.00417	-.00439	-.004.9	-.00447	-.00445	-.00457	-.00434
LOAD	4.75	5.96	5.48	5.44	6.23	6.79	6.71	5.42	5.30	5.79

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*              HOUSTON PILE GROUP STUDY             *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 2
CYCLE NO. 2100
LOAD ON GROUP = -46.95 FROM BIG LOAD CELL, KIPS
= -42.63 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.26444 INCHES
WEST CAP DEFL = -.27901 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-102.	-131.	-113.	-113.	-155.	-88.	-100.	-116.	-98.	-113.
24.	-144.	-189.	-171.	-154.	-194.	-176.	-132.	-173.	-121.	-152.
36.	-180.	-225.	-228.	-189.	-243.	-211.	-198.	-226.	-165.	-207.
48.	-265.	-249.	-266.	-211.	-272.	-27[illegible].	-222.	-256.	-187.	-244.
60.	-231.	-248.	-277.	-222.	-229.	-246.	-240.	-263.	-227.	-242.
72.	-210.	-228.	-265.	-217.	-220.	-251.	-224.	-247.	-261.	-237.
84.	-189.	-193.	-226.	-184.	-167.	-203.	-201.	-201.	-184.	-194.
96.	-137.	-131.	-171.	-145.	-136.	-148.	-151.	-144.	-129.	-144.
114.	-71.	-81.	-82.	-79.	-78.	-73.	-80.	-72.	-72.	-77.
132.	-27.	-39.	-21.	-28.	-34.	-29.	-37.	-23.	-29.	-30.
156.	-1.	-14.	6.	-0.	-6.	1.	-6.	1.	-3.	-2.
AT LOAD POINT:										
DEFL	-.25	-.23	-.25	-.24	-.24	-.25	-.25	-.26	-.27	-.25
SLOPE	.00351	.00298	.00357	.00328	.00344	.00330	.00321	.00364	.00322	.00337
LOAD	-4.98	-4.96	-4.47	-4.72	-5.93	-4.98	-4.11	-4.74	-3.75	-4.74

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*              HOUSTON PILE GROUP STUDY             *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 2
CYCLE NO. 1200
LOAD ON GROUP = 46.99 FROM BIG LOAD CELL, KIPS
= 48.36 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .32474 INCHES
WEST CAP DEFL = .28533 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	105.	112.	134.	144.	148.	179.	149.	129.	106.	134.
24.	148.	176.	191.	194.	196.	220.	218.	189.	169.	139.
36.	183.	222.	242.	242.	252.	264.	264.	244.	230.	239.
48.	234.	273.	287.	277.	325.	[illegible]	301.	284.	284.	286.
60.	298.	294.	320.	328.	362.	[illegible]	346.	315.	255.	317.
72.	318.	330.	324.	316.	334.	278.	337.	315.	300.	317.
84.	293.	268.	307.	275.	271.	234.	296.	286.	305.	282.
96.	242.	249.	262.	235.	227.	198.	230.	237.	193.	233.
114.	147.	139.	177.	141.	153.	117.	126.	142.	117.	140.
132.	88.	68.	94.	69.	33.	63.	66.	72.	67.	74.
156.	27.	14.	30.	23.	28.	25.	21.	24.	25.	24.
AT LOAD POINT:										
DEFL	.35	.34	.35	.32	.34	.32	.34	.35	.34	.34
SLOPE	-.00422	-.00458	-.00432	-.00419	-.00444	-.00422	-.00454	-.00458	-.00456	-.00442
LOAD	4.19	5.24	5.01	5.32	5.78	6.56	6.31	5.05	4.90	5.37

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*                  HOUSTON PILE GROUP STUDY                  *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984         *
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NORTH

D	G	A
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B	I	C

PILE LEGEND

LOAD NO. 2
CYCLE NO. 2200
LOAD ON GROUP = -41.97 FROM BIG LOAD CELL, KIPS
= -38.85 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.26391 INCHES
WEST CAP DEFL = -.27747 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-91.	-116.	-98.	-101.	-139.	-74.	-91.	-101.	-86.	-110.
24.	-128.	-165.	-147.	-140.	-175.	-158.	-119.	-151.	-135.	-143.
36.	-161.	-198.	-198.	-174.	-225.	-192.	-187.	-201.	-145.	-137.
48.	-244.	-225.	-238.	-200.	-259.	-252.	-209.	-238.	-165.	-226.
60.	-220.	-236.	-258.	-218.	-222.	-246.	-233.	-252.	-214.	-233.
72.	-265.	-219.	-251.	-214.	-215.	-255.	-231.	-241.	-254.	-232.
84.	-189.	-191.	-218.	-181.	-168.	-199.	-197.	-198.	-174.	-191.
96.	-137.	-135.	-170.	-143.	-133.	-145.	-149.	-140.	-123.	-142.
114.	-68.	-52.	-84.	-76.	-76.	-73.	-79.	-71.	-77.	-75.
132.	-24.	-39.	-22.	-28.	-33.	-3[illegible].	-36.	-23.	-29.	-29.
156.	0.	-13.	7.	-1.	-6.	-1.	-7.	0.	-2.	-2.
AT LOAD POINT:										
DEFL	-.24	-.22	-.24	-.23	-.23	-.24	-.24	-.25	-.26	-.24
SLOPE	.00337	.00279	.00332	.00322	.00331	.00334	.00309	.00337	.00304	.00321
LOAD	-4.66	-4.[illegible]4	-4.12	-4.25	-5.45	-4.59	-3.74	-4.15	-3.35	-4.32

HOUSTON PILE GROUP STUDY
FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984

	NORTH		
	D	G	A
	F	E	H
	B	I	C
	PILE LEGEND		

LOAD NO. 3
CYCLE NO. 1001
LOAD ON GROUP = 87.73 FROM BIG LOAD CELL, KIPS
= 89.41 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .56802 INCHES
WEST CAP DEFL = .53189 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	207.	215.	248.	275.	265.	284.	284.	241.	186.	245.
24.	288.	317.	340.	363.	345.	385.	376.	341.	304.	343.
36.	344.	374.	419.	475.	428.	482.	485.	427.	385.	424.
48.	426.	455.	479.	513.	533.	542.	550.	482.	431.	493.
60.	489.	471.	518.	528.	573.	542.	590.	517.	392.	514.
72.	481.	518.	526.	562.	562.	525.	546.	513.	457.	523.
84.	447.	452.	491.	497.	463.	467.	536.	464.	463.	475.
96.	357.	420.	427.	382.	372.	353.	372.	387.	327.	377.
114.	248.	246.	295.	238.	261.	216.	211.	238.	220.	241.
132.	137.	130.	161.	120.	150.	116.	118.	122.	121.	131.
156.	42.	33.	47.	39.	50.	41.	39.	38.	44.	42.
AT LOAD POINT:										
DEFL	.59	.60	.61	.56	.58	.57	.59	.59	.60	.59
SLOPE	-.00741	-.00755	-.00744	-.00754	-.00752	-.00748	-.00780	-.00778	-.00782	-.00759
LOAD	8.48	9.68	9.65	10.55	10.20	11.24	11.75	9.14	8.71	9.93

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*                 HOUSTON PILE GROUP STUDY                   *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984         *
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NORTH

D	G	A
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B	I	C

PILE LEGEND

LOAD NO. 3
CYCLE NO. 2001
LOAD ON GROUP = -80.35 FROM BIG LOAD CELL, KIPS
= -77.34 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.50549 INCHES
WEST CAP DEFL = -.52465 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-179.	-213.	-208.	-204.	-245.	-181.	-192.	-214.	-204.	-204.
24.	-242.	-306.	-303.	-273.	-311.	-309.	-238.	-306.	-250.	-282.
36.	-304.	-368.	-391.	-322.	-418.	-387.	-360.	-388.	-319.	-362.
48.	-409.	-414.	-468.	-377.	-472.	-442.	-434.	-437.	-380.	-426.
60.	-386.	-436.	-500.	-413.	-413.	-478.	-398.	-462.	-395.	-431.
72.	-388.	-470.	-473.	-450.	-430.	-507.	-476.	-454.	-439.	-454.
84.	-375.	-379.	-434.	-405.	-351.	-370.	-351.	-394.	-360.	-380.
96.	-287.	-323.	-346.	-315.	-324.	-349.	-327.	-306.	-295.	-317.
114.	-165.	-133.	-195.	-194.	-194.	-168.	-204.	-171.	-159.	-182.
132.	-83.	-93.	-75.	-87.	-96.	-82.	-97.	-71.	-70.	-84.
156.	-15.	-27.	-4.	-17.	-27.	-10.	-27.	-13.	-17.	-17.
AT LOAD POINT:										
DEFL	-.49	-.47	-.49	-.47	-.47	-.49	-.49	-.50	-.52	-.49
SLOPE	.00633	.00585	.00644	.00619	.00632	.00645	.00612	.00657	.00637	.00629
LOAD	-8.63	-8.56	-8.36	-8.19	-9.47	-9.22	-7.88	-8.62	-8.41	-8.59

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*                  HOUSTON PILE GROUP STUDY                    *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984            *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 3
CYCLE NO. 1005
LOAD ON GROUP = 78.19 FROM BIG LOAD CELL, KIPS
= 79.65 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .57010 INCHES
WEST CAP DEFL = .53141 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	185.	196.	220.	248.	230.	257.	256.	212.	164.	219.
24.	260.	293.	306.	331.	301.	352.	344.	302.	278.	307.
36.	316.	346.	380.	444.	379.	439.	461.	381.	358.	389.
48.	394.	422.	439.	485.	473.	520.	522.	430.	403.	454.
60.	459.	451.	476.	509.	525.	510.	586.	468.	359.	483.
72.	456.	495.	475.	545.	535.	501.	545.	482.	416.	494.
84.	439.	442.	455.	493.	455.	463.	543.	458.	433.	465.
96.	362.	404.	408.	383.	384.	343.	383.	392.	318.	375.
114.	259.	248.	309.	245.	271.	224.	209.	257.	227.	250.
132.	143.	133.	184.	125.	162.	123.	116.	136.	129.	139.
156.	45.	36.	60.	41.	54.	55.	37.	44.	48.	47.
AT LOAD POINT:										
DEFL	.59	.59	.60	.56	.58	.57	.59	.53	.58	.58
SLOPE	-.00707	-.00734	-.00703	-.00718	-.00723	-.00738	-.00754	-.00741	-.00748	-.00726
LOAD	7.67	8.90	9.54	9.38	8.78	10.19	10.50	8.02	7.66	8.35

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* HOUSTON PILE GROUP STUDY *
* FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
****

		NORTH
D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 3
CYCLE NO. 2205
LOAD ON GROUP = -73.79 FROM BIG LOAD CELL, KIPS
= -70.09 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.51843 INCHES
WEST CAP DEFL = -.53425 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-167.	-196.	-192.	-199.	-226.	-161.	-179.	-202.	-189.	-190.
24.	-227.	-283.	-279.	-269.	-290.	-279.	-223.	-292.	-235.	-264.
36.	-284.	-371.	-362.	-320.	-395.	-353.	-339.	-373.	-293.	-344.
48.	-383.	-391.	-436.	-377.	-441.	-394.	-419.	-424.	-362.	-403.
60.	-364.	-420.	-473.	-417.	-430.	-460.	-373.	-456.	-398.	-421.
72.	-364.	-468.	-473.	-450.	-426.	-505.	-456.	-455.	-432.	-448.
84.	-357.	-380.	-443.	-407.	-358.	-384.	-336.	-400.	-369.	-381.
96.	-295.	-339.	-367.	-317.	-308.	-373.	-318.	-316.	-310.	-327.
114.	-184.	-197.	-218.	-200.	-212.	-185.	-220.	-182.	-172.	-197.
132.	-104.	-106.	-91.	-93.	-106.	-94.	-111.	-79.	-78.	-96.
156.	-26.	-33.	-8.	-21.	-34.	-15.	-35.	-17.	-20.	-23.
AT LOAD POINT:										
DEFL	-.50	-.49	-.50	-.48	-.49	-.50	-.50	-.51	-.53	-.50
SLOPE	.00628	.00579	.00647	.00622	.00620	.00635	.00604	.00658	.00637	.00626
LOAD	-7.81	-7.67	-7.52	-7.86	-8.55	-8.02	-7.10	-7.95	-7.60	-7.79

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*                HOUSTON PILE GROUP STUDY              *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984   *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 3
CYCLE NO. 1010
LOAD ON GROUP = 73.02 FROM BIG LOAD CELL, KIPS
= 74.39 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .57069 INCHES
WEST CAP DEFL = .53159 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	168.	182.	208.	239.	221.	243.	247.	202.	157.	207.
24.	237.	271.	292.	320.	290.	330.	332.	290.	268.	292.
36.	288.	316.	365.	432.	368.	413.	458.	367.	351.	373.
48.	362.	391.	424.	473.	462.	496.	510.	417.	399.	437.
60.	430.	422.	464.	501.	514.	486.	580.	458.	351.	467.
72.	434.	463.	467.	538.	530.	483.	541.	475.	410.	482.
84.	427.	426.	451.	490.	451.	466.	537.	454.	430.	459.
96.	362.	403.	407.	386.	394.	345.	401.	394.	317.	379.
114.	280.	290.	314.	251.	267.	235.	213.	262.	229.	250.
132.	153.	147.	190.	130.	167.	132.	120.	142.	132.	145.
156.	50.	42.	64.	43.	57.	60.	38.	46.	51.	50.
AT LOAD POINT:										
DEFL	.59	.60	.60	.56	.58	.57	.59	.59	.58	.58
SLOPE	-.00687	-.00721	-.00692	-.00710	-.00714	-.00694	-.00748	-.00733	-.00739	-.00715
LOAD	6.79	8.13	7.95	8.89	8.24	9.29	9.96	7.60	7.24	8.23

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*               HOUSTON PILE GROUP STUDY            *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 3
CYCLE NO. 2010
LOAD ON GROUP = -72.87 FROM BIG LOAD CELL, KIPS
= -69.83 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.52130 INCHES
WEST CAP DEFL = -.53379 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-164.	-194.	-188.	-200.	-225.	-153.	-178.	-201.	-187.	-188.
24.	-224.	-280.	-275.	-271.	-289.	-268.	-222.	-290.	-234.	-262.
36.	-284.	-374.	-359.	-324.	-395.	-340.	-331.	-371.	-296.	-342.
48.	-360.	-390.	-434.	-382.	-442.	-375.	-420.	-423.	-357.	-400.
60.	-365.	[illegible]	-472.	-424.	-438.	-449.	-369.	-456.	-403.	-422.
72.	-366.	-475.	-478.	-457.	-431.	-504.	-457.	-458.	-431.	-451.
84.	-360.	-388.	-449.	-413.	-354.	-335.	-340.	-405.	-397.	-389.
96.	-305.	-345.	-376.	-318.	-308.	-385.	-312.	-322.	-310.	-331.
114.	-195.	[illegible]	-223.	-198.	-224.	-192.	-221.	-186.	-176.	-202.
132.	-105.	-105.	-92.	-91.	-106.	-93.	-111.	-81.	-77.	-96.
156.	-26.	-33.	-7.	-21.	-33.	-15.	-35.	-17.	-19.	-23.
AT LOAD POINT:										
DEFL	-.51	-.49	-.51	-.48	-.49	-.50	-.51	-.51	-.53	-.50
SLOPE	.00630	.00578	.00642	.00625	.00626	.00631	.00602	.00659	.00638	.00625
LOAD	-7.74	-7.63	-7.51	-7.97	-8.56	-7.83	-7.14	-7.92	-7.51	-7.76

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*                 HOUSTON PILE GROUP STUDY                   *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984         *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 3
CYCLE NO. 1020
LOAD ON GROUP = 72.30 FROM BIG LOAD CELL, KIPS
= 73.12 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .57250 INCHES
WEST CAP DEFL = .53228 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	161.	177.	203.	234.	219.	241.	244.	200.	155.	204.
24.	226.	264.	287.	314.	288.	327.	328.	288.	266.	289.
36.	275.	337.	358.	426.	366.	409.	459.	365.	350.	358.
48.	349.	393.	418.	466.	461.	491.	506.	417.	402.	433.
60.	416.	418.	460.	496.	514.	486.	582.	460.	357.	465.
72.	425.	457.	470.	540.	533.	484.	550.	478.	413.	483.
84.	425.	423.	455.	497.	450.	470.	539.	457.	434.	461.
96.	371.	402.	411.	396.	401.	350.	412.	398.	323.	385.
114.	294.	292.	320.	256.	257.	240.	217.	266.	232.	254.
132.	162.	150.	196.	131.	170.	136.	122.	145.	134.	150.
156.	55.	44.	68.	43.	59.	63.	39.	48.	53.	52.
AT LOAD POINT:										
DEFL	.60	.60	.61	.57	.58	.57	.59	.59	.58	.59
SLOPE	-.00683	-.00720	-.00693	-.00707	-.00714	-.00691	-.00747	-.00733	-.00735	-.00714
LOAD	6.49	8.01	7.81	8.77	8.23	9.11	9.86	7.61	7.16	8.12

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*              HOUSTON PILE GROUP STUDY          *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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			NORTH
	D	G	A
	F	E	H
	B	I	C
			PILE LEGEND

LOAD NO. 3
CYCLE NO. 2020
LOAD ON GROUP = -65.68 FROM BIG LOAD CELL, KIPS
= -62.85 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.52091 INCHES
WEST CAP DEFL = -.53271 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-150.	-177.	-170.	-180.	-209.	-135.	-163.	-183.	-170.	-171.
24.	-206.	-256.	-250.	-245.	-270.	-245.	-206.	-266.	-214.	-240.
36.	-266.	-345.	-328.	-293.	-375.	-313.	-309.	-342.	-272.	-316.
48.	-357.	-360.	-400.	-366.	-418.	-349.	-402.	-392.	-328.	-375.
60.	-348.	-392.	-439.	-386.	-420.	-425.	-351.	-427.	-389.	-397.
72.	-353.	-455.	-452.	-416.	-416.	-486.	-430.	-438.	-413.	-429.
84.	-352.	-379.	-432.	-385.	-363.	-375.	-332.	-396.	-392.	-379.
96.	-293.	-346.	-370.	-310.	-300.	-382.	-304.	-321.	-326.	-328.
114.	-195.	-236.	-231.	-214.	-237.	-194.	-220.	-193.	-180.	-213.
132.	-109.	-115.	-103.	-109.	-108.	-103.	-113.	-87.	-81.	-103.
156.	-28.	-37.	-11.	-29.	-35.	-18.	-36.	-20.	-20.	-26.
AT LOAD POINT:										
DEFL	-.50	-.48	-.50	-.48	-.49	-.50	-.50	-.51	-.53	-.50
SLOPE	.00607	.00551	.00615	.00598	.00601	.00603	.00581	.00640	.00617	.00601
LOAD	-7.01	-6.81	-6.73	-7.14	-7.68	-6.98	-6.46	-7.12	-6.72	-6.98

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*****************************************************
*              HOUSTON PILE GROUP STUDY             *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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                                                         NORTH
LOAD NO.  3                                            D   G   A
CYCLE NO.  1050
LOAD ON GROUP =   68.65 FROM BIG LOAD CELL, KIPS       F   E   H
              =   69.13 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL =   .57926 INCHES                        B   I   C
WEST CAP DEFL =   .53383 INCHES
                                                       PILE LEGEND
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DEPTH, INCHES	BENDING MOMENTS, INCH-KIPS									
	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	148.	169.	191.	219.	211.	226.	232.	192.	147.	193.
24.	209.	254.	269.	294.	278.	309.	313.	277.	257.	273.
36.	257.	296.	340.	399.	354.	393.	433.	353.	344.	352.
48.	331.	371.	398.	440.	450.	449.	484.	407.	401.	415.
60.	402.	412.	445.	467.	512.	475.	562.	452.	356.	454.
72.	416.	446.	469.	510.	538.	472.	543.	478.	411.	476.
84.	421.	423.	457.	481.	444.	471.	530.	463.	433.	453.
96.	380.	407.	420.	443.	409.	360.	437.	408.	328.	399.
114.	333.	316.	333.	278.	257.	240.	251.	277.	236.	280.
132.	186.	158.	207.	149.	178.	137.	132.	154.	135.	160.
156.	58.	48.	71.	50.	60.	53.	42.	52.	52.	55.
AT LOAD POINT:										
DEFL	.61	.61	.62	.57	.59	.58	.60	.60	.59	.60
SLOPE	-.00678	-.00721	-.00689	-.00695	-.00715	-.00686	-.00743	-.00737	-.00733	-.00711
LOAD	5.89	7.54	7.28	8.21	7.90	8.72	9.36	7.36	6.83	7.68

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*              HOUSTON PILE GROUP STUDY                 *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984    *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 3
CYCLE NO. 2050
LOAD ON GROUP = -61.48 FROM BIG LOAD CELL, KIPS
= -59.0? FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.52473 INCHES
WEST CAP DEFL = -.53186 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-139.	-165.	-158.	-172.	-203.	-124.	-155.	-176.	-159.	-161.
24.	-190.	-241.	-234.	-235.	-263.	-228.	-196.	-255.	-201.	-227.
36.	-248.	-325.	-308.	-284.	-366.	-289.	-305.	-329.	-255.	-301.
48.	-336.	-342.	-379.	-357.	-410.	-353.	-393.	-380.	-304.	-352.
60.	-337.	-422.	-419.	-382.	-418.	-412.	-342.	-419.	-381.	-390.
72.	-348.	-445.	-436.	-416.	-415.	-485.	-422.	-435.	-405.	-423.
84.	-358.	-377.	-426.	-393.	-378.	-381.	-351.	-397.	-401.	-335.
96.	-302.	-347.	-376.	-310.	-304.	-388.	-299.	-326.	-343.	-333.
114.	-203.	-215.	-247.	-217.	-245.	-209.	-224.	-200.	-191.	-216.
132.	-113.	-119.	-117.	-111.	-109.	-115.	-116.	-93.	-90.	-109.
156.	-32.	-38.	-18.	-30.	-36.	-24.	-39.	-23.	-25.	-33.
AT LOAD POINT:										
DEFL	-.50	-.46	-.50	-.47	-.49	-.50	-.50	-.51	-.53	-.50
SLOPE	.00599	.00539	.00607	.00598	.00598	.00596	.00576	.00631	.00608	.00595
LOAD	-6.43	-6.42	-6.31	-5.80	-7.61	-6.39	-6.11	-5.75	-6.19	-6.36

HOUSTON PILE GROUP STUDY

FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 3
CYCLE NO. 1100
LOAD ON GROUP = 67.00 FROM BIG LOAD CELL, KIPS
= 66.84 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .58278 INCHES
WEST CAP DEFL = .53514 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	139.	164.	188.	202.	202.	213.	219.	184.	141.	133.
24.	197.	247.	254.	274.	269.	299.	299.	267.	250.	252.
36.	245.	285.	323.	373.	342.	39[illegible].	412.	343.	338.	340.
48.	320.	355.	380.	409.	436.	436.	466.	398.	398.	401.
60.	394.	413.	432.	444.	507.	470.	546.	446.	352.	445.
72.	413.	445.	462.	491.	539.	456.	528.	478.	410.	469.
84.	422.	42[illegible].	460.	473.	478.	47[illegible].	527.	468.	438.	453.
96.	387.	415.	431.	454.	414.	357.	431.	418.	335.	405.
114.	369.	336.	347.	289.	285.	248.	267.	288.	240.	297.
132.	194.	15[illegible].	219.	163.	169.	138.	148.	162.	135.	158.
156.	60.	54.	76.	55.	61.	62.	43.	54.	52.	58.
AT LOAD POINT:										
DEFL	.62	.61	.63	.58	.60	.59	.61	.62	.60	.61
SLOPE	-.00687	-.00731	-.00694	-.00689	-.00721	-.00693	-.00747	-.00743	-.00733	-.00715
LOAD	5.53	7.29	6.91	7.8[illegible]	7.78	8.[illegible]1	9.07	7.2[illegible]	6.62	7.43

HOUSTON PILE GROUP STUDY

FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 3
CYCLE NO. 2100
LOAD ON GROUP = -61.11 FROM BIG LOAD CELL, KIPS
= -59.19 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.52502 INCHES
WEST CAP DEFL = -.52999 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-132.	-159.	-149.	-170.	-207.	-124.	-152.	-174.	-152.	-153.
24.	-182.	-230.	-221.	-232.	-257.	-221.	-192.	-252.	-192.	-221.
36.	-236.	-316.	-290.	-283.	-371.	-273.	-299.	-324.	-243.	-293.
48.	-319.	-335.	-359.	-357.	-414.	-349.	-383.	-375.	-293.	-354.
60.	-326.	-401.	-402.	-385.	-425.	-409.	-341.	-418.	-375.	-387.
72.	-346.	-443.	-428.	-423.	-423.	-490.	-430.	-438.	-403.	-425.
84.	-368.	-332.	-426.	-410.	-355.	-386.	-360.	-402.	-404.	-338.
96.	-314.	-350.	-386.	-310.	-306.	-393.	-318.	-332.	-352.	-340.
114.	-198.	-197.	-262.	-217.	-216.	-217.	-223.	-205.	-202.	-215.
132.	-118.	-114.	-127.	-108.	-108.	-121.	-110.	-95.	-97.	-111.
156.	-33.	-34.	-20.	-29.	-39.	-27.	-41.	-23.	-26.	-30.
AT LOAD POINT:										
DEFL	-.50	-.48	-.50	-.47	-.48	-.49	-.50	-.51	-.52	-.50
SLOPE	.00599	.00531	.00601	.00603	.00602	.00590	.00574	.00632	.00535	.00593
LOAD	-6.38	-6.50	-6.23	-6.84	-7.89	-6.43	-6.14	-6.80	-5.98	-6.58

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*                HOUSTON PILE GROUP STUDY              *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984   *
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NORTH

LOAD NO. 3
CYCLE NO. 1200
LOAD ON GROUP = 63.95 FROM BIG LOAD CELL, KIPS
= 62.77 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = .58964 INCHES
WEST CAP DEFL = .53857 INCHES

D	G	A
F	E	H
B	I	C

PILE LEGEND

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	129.	157.	164.	184.	192.	205.	205.	175.	133.	172.
24.	184.	237.	235.	250.	258.	286.	281.	257.	241.	243.
36.	231.	277.	301.	345.	328.	378.	387.	333.	329.	323.
48.	300.	353.	357.	381.	424.	410.	440.	391.	391.	334.
60.	380.	407.	412.	421.	501.	462.	521.	441.	347.	432.
72.	407.	443.	449.	470.	538.	447.	511.	477.	409.	461.
84.	421.	412.	459.	464.	481.	487.	526.	471.	445.	453.
96.	394.	424.	442.	470.	421.	370.	440.	426.	348.	415.
114.	379.	356.	365.	307.	306.	261.	292.	299.	250.	313.
132.	205.	177.	235.	180.	192.	149.	164.	171.	143.	130.
156.	65.	60.	83.	61.	64.	68.	50.	58.	55.	63.
AT LOAD POINT:										
DEFL	.63	.63	.64	.60	.61	.60	.62	.63	.61	.62
SLOPE	-.00689	-.00737	-.00695	-.00675	-.00724	-.00695	-.00745	-.00745	-.00731	-.00715
LOAD	5.02	6.68	5.24	7.24	7.49	8.18	8.53	6.95	6.27	6.97

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*             HOUSTON PILE GROUP STUDY              *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 3
CYCLE NO. 2200
LOAD ON GROUP = -52.17 FROM BIG LOAD CELL, KIPS
= -51.74 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.52397 INCHES
WEST CAP DEFL = -.52541 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-114.	-140.	-128.	-148.	-193.	-108.	-137.	-153.	-133.	-139.
24.	-158.	-232.	-191.	-203.	-250.	-198.	-175.	-223.	-168.	-196.
36.	-207.	-281.	-252.	-249.	-351.	-244.	-282.	-290.	-214.	-253.
48.	-281.	-299.	-315.	-318.	-393.	-322.	-359.	-338.	-256.	-320.
60.	-295.	-361.	-360.	-355.	-412.	-379.	-325.	-385.	-344.	-357.
72.	-323.	-411.	-397.	-400.	-417.	-473.	-420.	-419.	-372.	-404.
84.	-360.	-335.	-407.	-411.	-351.	-371.	-357.	-396.	-387.	-331.
96.	-315.	-346.	-381.	-307.	[illegible]	-383.	-320.	-333.	-349.	-337.
114.	-213.	[illegible]	-275.	-222.	-253.	-223.	-217.	-213.	-214.	-220.
132.	-127.	-120.	-142.	-117.	-117.	-125.	-109.	-103.	-138.	-119.
156.	-38.	-35.	-27.	-34.	-46.	-31.	-42.	-28.	-31.	-34.
AT LOAD POINT:										
DEFL	-.50	-.46	-.49	-.46	-.47	-.49	-.49	-.50	-.51	-.49
SLOPE	.00559	.00496	.00562	.00577	.00571	.00549	.00545	.00602	.00574	.00559
LOAD	-5.50	-5.71	-5.35	-5.84	-7.25	-5.56	-5.44	-5.88	-5.11	-5.75

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*            HOUSTON PILE GROUP STUDY              *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
****************************************************
```

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 4
CYCLE NO. 1001
LOAD ON GROUP = 139.14 FROM BIG LOAD CELL, KIPS
= 138.79 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = 1.27664 INCHES
WEST CAP DEFL = 1.24745 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	340.	347.	374.	458.	403.	396.	466.	393.	273.	383.
24.	526.	548.	522.	541.	563.	635.	639.	557.	428.	552.
36.	573.	632.	657.	769.	639.	683.	781.	700.	540.	664.
48.	673.	731.	750.	860.	829.	785.	919.	798.	637.	776.
60.	762.	797.	841.	941.	904.	850.	1024.	879.	597.	844.
72.	801.	873.	881.	993.	987.	857.	997.	935.	693.	891.
84.	790.	839.	883.	942.	930.	899.	996.	911.	734.	831.
96.	785.	632.	855.	887.	899.	813.	846.	841.	635.	822.
114.	670.	696.	731.	537.	562.	557.	617.	626.	589.	654.
132.	431.	460.	502.	362.	467.	456.	376.	385.	398.	425.
156.	143.	159.	194.	137.	169.	184.	128.	138.	173.	159.
AT LOAD POINT:										
DEFL	1.31	1.34	1.33	1.29	1.30	1.31	1.32	1.32	1.29	1.31
SLOPE	-.01501	-.01467	-.01476	-.01530	-.01480	-.01478	-.01572	-.01548	-.01438	-.01499
LOAD	13.60	15.03	14.91	13.09	15.21	16.18	19.39	14.48	11.92	15.42

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*                  HOUSTON PILE GROUP STUDY                   *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984          *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 4
CYCLE NO. 2001
LOAD ON GROUP = -127.88 FROM BIG LOAD CELL, KIPS
= -125.60 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -1.25472 INCHES
WEST CAP DEFL = -1.27966 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-281.	-349.	-389.	-327.	-381.	-324.	-305.	-359.	-376.	-343.
24.	-419.	-521.	-569.	-487.	-537.	-477.	-451.	-516.	-534.	-498.
36.	-531.	-649.	-733.	-597.	-555.	-608.	-584.	-655.	-623.	-626.
48.	-611.	-753.	-853.	-703.	-778.	-745.	-694.	-745.	-694.	-732.
60.	-659.	-824.	-919.	-891.	-825.	-837.	-716.	-819.	-825.	-810.
72.	-708.	-865.	-953.	-822.	-870.	-934.	-823.	-879.	-859.	-857.
84.	-734.	-853.	-963.	-832.	-845.	-868.	-827.	-884.	-837.	-850.
96.	-696.	-786.	-864.	-753.	-813.	-839.	-737.	-814.	-749.	-784.
114.	-583.	-631.	-657.	-548.	-664.	-620.	-610.	-616.	-521.	-617.
132.	-424.	-396.	-384.	-463.	-446.	-423.	-472.	-367.	-333.	-439.
156.	-177.	-126.	-108.	-157.	-170.	-145.	-196.	-123.	-92.	-143.
AT LOAD POINT:										
DEFL	-1.24	-1.25	-1.25	-1.22	-1.22	-1.26	-1.27	-1.25	-1.29	-1.25
SLOPE	.01352	.01325	.01413	.01359	.01355	.01377	.01314	.01444	.01480	.01379
LOAD	-12.98	-14.20	-15.27	-13.64	-14.38	-14.59	-12.41	-13.98	-14.95	-13.93

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* HOUSTON PILE GROUP STUDY *

* FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *

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LOAD NO. 4
CYCLE NO. 1005
LOAD ON GROUP = 124.67 FROM BIG LOAD CELL, KIPS
= 125.19 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = 1.34280 INCHES
WEST CAP DEFL = 1.29472 INCHES

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	305.	299.	325.	441.	370.	355.	418.	355.	254.	347.
24.	489.	480.	466.	589.	534.	591.	586.	513.	396.	515.
36.	538.	568.	599.	727.	604.	647.	735.	654.	517.	621.
48.	645.	646.	595.	830.	783.	753.	868.	761.	616.	733.
60.	736.	730.	789.	916.	886.	826.	979.	849.	585.	811.
72.	785.	812.	833.	989.	966.	847.	977.	918.	703.	870.
84.	787.	790.	877.	963.	925.	905.	1005.	913.	737.	878.
96.	794.	786.	843.	924.	903.	813.	888.	869.	660.	831.
114.	707.	725.	765.	712.	702.	698.	665.	678.	599.	695.
132.	476.	570.	564.	393.	509.	505.	421.	435.	422.	477.
156.	159.	216.	235.	155.	191.	214.	139.	160.	200.	135.
AT LOAD POINT:										
DEFL	1.36	1.38	1.39	1.35	1.36	1.37	1.38	1.37	1.33	1.37
SLOPE	-.01501	-.01528	-.01456	-.01532	-.01521	-.01474	-.01566	-.01547	-.01460	-.01509
LOAD	12.34	13.06	13.06	15.36	14.01	14.86	17.32	13.24	10.93	13.91

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**********************************************************
*              HOUSTON PILE GROUP STUDY                  *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984     *
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```

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 4
CYCLE NO. 2005
LOAD ON GROUP = -123.12 FROM BIG LOAD CELL, KIPS
= -120.79 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -1.26839 INCHES
WEST CAP DEFL = -1.28453 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-267.	-328.	-365.	-308.	-366.	-307.	-290.	-341.	-357.	-325.
24.	-403.	-493.	-537.	-467.	-521.	-458.	-437.	-494.	-486.	-477.
36.	-517.	-620.	-702.	-584.	-643.	-593.	-569.	-639.	-613.	-613.
48.	-604.	-744.	-830.	-594.	-773.	-743.	-680.	-734.	-687.	-721.
60.	-658.	-808.	-895.	-793.	-819.	-830.	-705.	-806.	-832.	-791.
72.	-706.	-848.	-924.	-812.	-863.	-925.	-810.	-865.	-855.	-845.
84.	-733.	-849.	-937.	-825.	-542.	-864.	-821.	-874.	-840.	-843.
96.	-701.	-786.	-854.	-752.	-816.	-841.	-743.	-816.	-755.	-785.
114.	-594.	-647.	-689.	-652.	-690.	-630.	-623.	-538.	-544.	-634.
132.	-439.	-413.	-426.	-497.	-461.	-442.	-486.	-394.	-318.	-431.
156.	-189.	-135.	-128.	-169.	-182.	-155.	-213.	-138.	-102.	-157.
AT LOAD POINT:										
DEFL	-1.26	-1.25	-1.27	-1.22	-1.23	-1.27	-1.28	-1.27	-1.30	-1.25
SLOPE	.01356	.01317	.01467	.01356	.01356	.01367	.01308	.01435	.01472	.01375
LOAD	-12.41	-13.57	-14.54	-12.65	-13.95	-14.12	-11.91	-13.35	-14.28	-13.42

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*              HOUSTON PILE GROUP STUDY                   *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984      *
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```

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 4
CYCLE NO. 1010
LOAD ON GROUP = 120.58 FROM BIG LOAD CELL, KIPS
= 120.32 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = 1.34948 INCHES
WEST CAP DEFL = 1.29333 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	291.	255.	307.	424.	353.	334.	401.	333.	244.	330.
24.	470.	456.	444.	554.	511.	559.	565.	483.	379.	492.
36.	518.	543.	572.	599.	583.	621.	715.	618.	501.	597.
48.	627.	616.	567.	815.	751.	719.	846.	725.	502.	703.
60.	721.	707.	763.	891.	867.	[illegible]	961.	815.	575.	789.
72.	773.	793.	869.	968.	953.	823.	964.	889.	695.	853.
84.	779.	758.	561.	947.	922.	893.	997.	893.	728.	849.
96.	790.	752.	837.	925.	926.	808.	889.	861.	661.	829.
114.	714.	731.	766.	734.	715.	711.	680.	597.	598.	705.
132.	489.	601.	582.	406.	523.	526.	465.	465.	427.	498.
156.	171.	242.	252.	164.	199.	223.	145.	177.	205.	198.
AT LOAD POINT:										
DEFL	1.38	1.38	1.40	1.36	1.37	1.38	1.39	1.38	1.34	1.33
SLOPE	-.01497	-.01539	-.01444	-.01526	-.01505	-.01465	-.01563	-.01532	-.01459	-.01503
LOAD	11.90	12.49	12.46	15.77	13.52	14.11	16.75	12.55	13.45	13.34

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*             HOUSTON PILE GROUP STUDY           *
* FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 4
CYCLE NO. 2110
LOAD ON GROUP = -113.14 FROM BIG LOAD CELL, KIPS
= -111.31 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -1.27318 INCHES
WEST CAP DEFL = -1.28222 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-248.	-306.	-342.	-289.	-343.	-285.	-271.	-319.	-339.	-305.
24.	-379.	-451.	-507.	-443.	-492.	-432.	-417.	-463.	-453.	-451.
36.	-490.	-578.	-660.	-549.	-608.	-555.	-536.	-595.	-530.	-572.
48.	-567.	-693.	-781.	-651.	-730.	-696.	-636.	-688.	-654.	-677.
60.	-624.	-759.	-849.	-751.	-779.	[illegible]	-669.	-765.	-774.	-751.
72.	-676.	-836.	-887.	-778.	-832.	-891.	-779.	-831.	-834.	-813.
84.	-711.	-816.	-913.	-803.	-822.	-847.	-803.	-851.	-834.	-822.
96.	-690.	-772.	-843.	-738.	-816.	-829.	-738.	-808.	-751.	-775.
114.	-594.	-647.	-700.	-649.	-697.	-636.	-623.	-648.	-555.	-639.
132.	-450.	-420.	-446.	-493.	-474.	-459.	-497.	-414.	-339.	-443.
156.	-200.	-140.	-141.	-180.	-192.	-164.	-224.	-150.	-136.	-165.
AT LOAD POINT:										
DEFL	-1.27	-1.25	-1.27	-1.22	-1.23	-1.26	-1.28	-1.27	-1.30	-1.26
SLOPE	.01330	.01289	.01381	.01330	.01332	.01343	.01285	.01413	.01446	.01353
LOAD	-11.35	-12.48	-13.42	-11.73	-12.86	-12.98	-11.00	-12.23	-13.26	-12.37

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* HOUSTON PILE GROUP STUDY *
* FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
****

	NORTH	
D	G	A
F	E	H
B	I	C

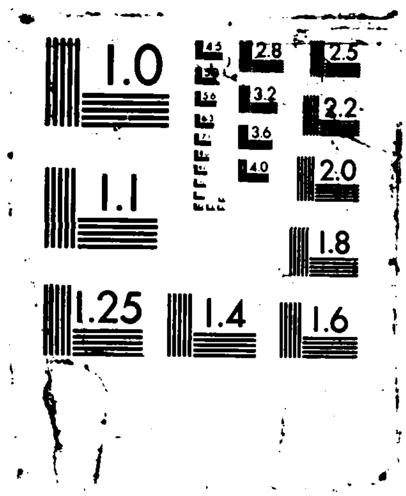
PILE LEGEND

LOAD NO. 4
CYCLE NO. 1020
LOAD ON GROUP = 110.83 FROM BIG LOAD CELL, KIPS
= 109.22 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = 1.37545 INCHES
WEST CAP DEFL = 1.31270 INCHES

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
	BENDING MOMENTS, INCH-KIPS									
12.	278.	255.	293.	413.	339.	322.	398.	319.	251.	323.
24.	452.	427.	431.	546.	494.	543.	558.	465.	383.	477.
36.	497.	517.	555.	580.	564.	607.	706.	598.	508.	581.
48.	605.	593.	653.	787.	74[illegible].	713.	841.	705.	612.	694.
60.	703.	694.	748.	876.	851.	785.	953.	801.	593.	777.
72.	763.	782.	803.	956.	947.	822.	964.	879.	712.	847.
84.	775.	779.	860.	940.	929.	894.	1000.	893.	747.	863.
96.	798.	778.	849.	933.	923.	823.	899.	874.	694.	841.
114.	737.	737.	796.	763.	746.	744.	704.	730.	632.	732.
132.	516.	619.	515.	468.	554.	554.	502.	504.	467.	533.
156.	195.	259.	278.	197.	227.	257.	170.	203.	233.	223.
AT LOAD POINT:										
DEFL	1.40	1.40	1.44	1.39	1.40	1.42	1.42	1.42	1.40	1.41
SLOPE	-.01492	-.01543	-.01438	-.01521	-.01494	-.01453	-.01570	-.01527	-.01517	-.01507
LOAD	10.89	10.96	11.13	14.67	12.51	12.92	15.83	11.84	3.48	12.14

AD-A193 498 BEHAVIOR OF A LARGE-SCALE PILE GROUP SUBJECTED TO CYCLIC LATERAL LOADING..(U) TEXAS UNIV AT AUSTIN GEOTECHNICAL ENGINEERING CENTER D A BROWN ET AL. 5/5

UNCLASSIFIED FEB 88 WES/MP/GL-88-2 DACW39-83-C-0061 F/G 13/13 NL


1.0
1.1
1.25
1.4
1.6
1.8
2.0
2.2
2.5
2.8
3.2
3.6
4.0

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*                 HOUSTON PILE GROUP STUDY                   *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984         *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 4
CYCLE NO. 2020
LOAD ON GROUP = -108.69 FROM BIG LOAD CELL, KIPS
= -104.14 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -1.27494 INCHES
WEST CAP DEFL = -1.28218 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-235.	-295.	-324.	-270.	-319.	-263.	-248.	-300.	-332.	-287.
24.	-362.	-450.	-479.	-425.	-454.	-434.	-392.	-439.	-457.	-431.
36.	-473.	-560.	-628.	-529.	-577.	-521.	-507.	-568.	-574.	-549.
48.	-548.	-675.	-746.	-628.	-697.	-654.	-598.	-661.	-650.	-651.
60.	-605.	-746.	-817.	-729.	-747.	-758.	-637.	-737.	-774.	-723.
72.	-655.	-792.	-855.	-757.	-834.	-860.	-747.	-808.	-842.	-791.
84.	-695.	-812.	-887.	-792.	-799.	-826.	-779.	-838.	-847.	-823.
96.	-681.	-753.	-850.	-732.	-792.	-813.	-731.	-801.	-765.	-772.
114.	-593.	-663.	-706.	-650.	-697.	-634.	-626.	-651.	-577.	-644.
132.	-462.	-440.	-465.	-495.	-485.	-465.	-497.	-424.	-352.	-454.
156.	-206.	-15[illegible].	-149.	-184.	-193.	-163.	-225.	-156.	-136.	-172.
AT LOAD POINT:										
DEFL	-1.27	-1.25	-1.27	-1.22	-1.23	-1.26	-1.28	-1.27	-1.35	-1.26
SLOPE	.01323	.01284	.01380	.01329	.01326	.01334	.01272	.01405	.01482	.01343
LOAD	-10.88	-12.00	-12.84	-11.27	-12.22	-12.35	-10.43	-11.61	-10.53	-11.57

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**********************************************************
*                HOUSTON PILE GROUP STUDY                *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984     *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 4
CYCLE NO. 1050
LOAD ON GROUP = 105.10 FROM BIG LOAD CELL, KIPS
= 105.57 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = 1.36791 INCHES
WEST CAP DEFL = 1.32584 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	249.	247.	272.	387.	317.	331.	377.	301.	240.	302.
24.	406.	399.	399.	507.	459.	503.	525.	440.	367.	445.
36.	445.	487.	515.	635.	527.	575.	669.	568.	490.	545.
48.	544.	558.	604.	739.	693.	668.	798.	674.	589.	652.
60.	633.	649.	697.	825.	803.	745.	906.	772.	576.	734.
72.	694.	751.	755.	905.	900.	781.	918.	853.	692.	805.
84.	715.	751.	815.	898.	899.	863.	963.	877.	738.	835.
96.	758.	762.	829.	917.	914.	811.	880.	877.	699.	828.
114.	746.	746.	800.	779.	767.	752.	720.	746.	640.	744.
132.	554.	627.	670.	515.	584.	576.	536.	527.	475.	563.
156.	232.	274.	305.	213.	258.	272.	192.	218.	242.	245.
AT LOAD POINT:										
DEFL	1.41	1.45	1.45	1.40	1.41	1.43	1.43	1.43	1.41	1.42
SLOPE	-.01456	-.01450	-.01422	-.01485	-.01468	-.01433	-.01545	-.01524	-.01501	-.01476
LOAD	9.75	10.31	10.34	13.87	11.70	12.19	15.15	11.39	10.35	11.73

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*                HOUSTON PILE GROUP STUDY                *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984     *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 4
CYCLE NO. 2050
LOAD ON GROUP = -99.26 FROM BIG LOAD CELL, KIPS
= -96.74 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -1.27476 INCHES
WEST CAP DEFL = -1.28068 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-211.	-255.	-292.	-244.	-288.	-21[illegible].	-22[illegible].	-272.	-299.	-255.
24.	-326.	-4[illegible]4.	-434.	-390.	-426.	-372.	-358.	-400.	-413.	-391.
36.	-429.	-501.	-569.	-488.	-532.	-48[illegible].	-463.	-521.	-521.	-5[illegible].
48.	-495.	-6[illegible].	-684.	-582.	-644.	-6[illegible]9.	-546.	-610.	-595.	-597.
60.	-555.	-678.	-751.	-684.	-694.	-7[illegible].	-686.	-687.	-719.	-674.
72.	-609.	-72[illegible].	-794.	-715.	-759.	-815.	-697.	-762.	-794.	-742.
84.	-657.	-75[illegible].	-832.	-752.	-764.	-799.	-741.	-801.	-811.	-77[illegible].
96.	-662.	-75[illegible].	-808.	-713.	-774.	-794.	-719.	-781.	-749.	-75[illegible].
114.	-600.	-660.	-710.	-653.	-702.	-64[illegible].	-631.	-636.	-592.	-65[illegible].
132.	-496.	-4[illegible]1.	-503.	-507.	-504.	-489.	-517.	-445.	-402.	-433.
156.	-228.	-17[illegible].	-181.	-195.	-205.	-179.	-241.	-173.	-125.	-189.
AT LOAD POINT:										
DEFL	-1.27	-1.24	-1.26	-1.21	-1.22	-1.25	-1.27	-1.26	-1.34	-1.26
SLOPE	.01278	.01233	.01330	.01295	.01284	.01294	.01231	.01361	.01434	.01334
LOAD	-9.84	-10.98	-11.82	-10.42	-11.21	-11.35	-9.47	-10.58	-11.08	-10.75

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*                 HOUSTON PILE GROUP STUDY                *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984      *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 4
CYCLE NO. 1100
LOAD ON GROUP = 96.35 FROM BIG LOAD CELL, KIPS
= 95.87 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = 1.38116 INCHES
WEST CAP DEFL = 1.32437 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	221.	227.	247.	361.	280.	295.	351.	280.	224.	276.
24.	364.	358.	362.	471.	409.	463.	488.	410.	345.	419.
36.	398.	450.	471.	593.	471.	517.	632.	533.	465.	513.
48.	491.	516.	555.	699.	621.	623.	753.	536.	560.	596.
60.	579.	635.	646.	785.	731.	704.	861.	733.	552.	689.
72.	643.	739.	736.	870.	834.	748.	882.	821.	663.	755.
84.	672.	719.	775.	872.	851.	835.	935.	855.	720.	814.
96.	721.	738.	799.	913.	881.	795.	867.	868.	691.	818.
114.	745.	751.	813.	788.	783.	755.	734.	763.	642.	752.
132.	584.	659.	717.	545.	625.	596.	556.	557.	485.	593.
156.	271.	292.	339.	234.	284.	286.	209.	239.	253.	258.
AT LOAD POINT:										
DEFL	1.43	1.47	1.47	1.41	1.43	1.44	1.44	1.45	1.42	1.44
SLOPE	-.01424	-.01432	-.01397	-.01458	-.01431	-.01417	-.01527	-.01502	-.01486	-.01453
LOAD	8.50	9.20	9.25	12.95	10.43	11.13	14.10	10.61	9.79	10.65

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*               HOUSTON PILE GROUP STUDY                 *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984     *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 4
CYCLE NO. 2100
LOAD ON GROUP = -90.45 FROM BIG LOAD CELL, KIPS
= -88.63 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -1.28312 INCHES
WEST CAP DEFL = -1.28349 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-185.	-241.	-267.	-218.	-260.	-196.	-201.	-250.	-277.	-233.
24.	-287.	-356.	-395.	-358.	-387.	-342.	-333.	-366.	-384.	-353.
36.	-380.	-453.	-518.	-449.	-455.	-460.	-428.	-478.	-487.	-453.
48.	-438.	-550.	-624.	-539.	-589.	-569.	-507.	-563.	-561.	-549.
60.	-499.	-614.	-688.	-542.	-636.	-666.	-548.	-634.	-681.	-623.
72.	-553.	-666.	-729.	-676.	-707.	-772.	-659.	-710.	-759.	-692.
84.	-611.	-717.	-769.	-737.	-728.	-773.	-710.	-758.	-785.	-731.
96.	-635.	-714.	-759.	-695.	-751.	-784.	-706.	-759.	-737.	-728.
114.	-605.	-658.	-701.	-671.	-718.	-653.	-641.	-608.	-637.	-653.
132.	-534.	-510.	-551.	-536.	-539.	-509.	-539.	-478.	-427.	-513.
156.	-259.	-215.	-223.	-212.	-226.	-193.	-257.	-199.	-141.	-213.
AT LOAD POINT:										
DEFL	-1.28	-1.24	-1.26	-1.21	-1.22	-1.25	-1.28	-1.27	-1.34	-1.26
SLOPE	.01240	.01190	.01291	.01271	.01252	.01265	.01204	.01332	.01412	.01273
LOAD	-8.74	-10.04	-10.94	-9.54	-10.13	-10.49	-8.63	-9.68	-10.55	-9.87

HOUSTON PILE GROUP STUDY

FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 4
CYCLE NO. 1200
LOAD ON GROUP = 85.41 FROM BIG LOAD CELL, KIPS
= 84.50 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = 1.38354 INCHES
WEST CAP DEFL = 1.32873 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	195.	233.	219.	311.	249.	255.	303.	247.	198.	243.
24.	324.	334.	322.	403.	367.	429.	419.	365.	337.	353.
36.	355.	411.	422.	510.	423.	473.	550.	476.	422.	433.
48.	448.	472.	502.	510.	566.	584.	658.	572.	510.	547.
60.	538.	563.	592.	592.	561.	673.	756.	566.	504.	629.
72.	627.	668.	555.	785.	791.	727.	795.	757.	613.	711.
84.	643.	658.	729.	814.	824.	826.	869.	804.	674.	754.
96.	695.	719.	773.	895.	868.	791.	835.	842.	673.	736.
114.	739.	752.	812.	830.	767.	752.	757.	781.	647.	752.
132.	632.	654.	736.	612.	635.	651.	609.	602.	511.	621.
156.	308.	315.	373.	271.	305.	293.	244.	274.	284.	297.
AT LOAD POINT:										
DEFL	1.44	1.48	1.49	1.42	1.44	1.45	1.45	1.45	1.43	1.45
SLOPE	-.01395	-.01412	-.01374	-.01401	-.01405	-.01399	-.01472	-.01461	-.01443	-.01413
LOAD	7.38	8.18	8.09	11.12	9.38	10.05	12.24	9.45	8.51	9.39

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*                HOUSTON PILE GROUP STUDY                  *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984       *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 4
CYCLE NO. 2200
LOAD ON GROUP = -103.99 FROM RIG LOAD CELL, KIPS
= -78.47 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -1.27978 INCHES
WEST CAP DEFL = -1.27644 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-156.	-212.	-235.	-189.	-235.	-189.	-175.	-221.	-241.	-236.
24.	-244.	-318.	-350.	-317.	-349.	-313.	-297.	-324.	-337.	-317.
36.	-329.	-396.	-462.	-431.	-441.	-43[illegible].	-381.	-423.	-429.	-410.
48.	-378.	-451.	-563.	-482.	-537.	-535.	-451.	-499.	-502.	-492.
60.	-441.	-536.	-622.	-586.	-586.	-631.	-488.	-566.	-618.	-553.
72.	-498.	-580.	-670.	-624.	-657.	-742.	-603.	-640.	-693.	-634.
84.	-563.	-622.	-715.	-698.	-692.	-759.	-667.	-700.	-727.	-683.
96.	-610.	-652.	-719.	-690.	-740.	-776.	-691.	-727.	-704.	-703.
114.	-604.	-643.	-700.	-674.	-718.	-650.	-653.	-676.	-616.	-661.
132.	-563.	-530.	-587.	-557.	-563.	-513.	-567.	-517.	-474.	-543.
156.	-282.	-239.	-262.	-233.	-251.	-2[illegible].	-284.	-234.	-177.	-241.
AT LOAD POINT:										
DEFL	-1.28	-1.22	-1.25	-1.22	-1.21	-1.24	-1.27	-1.25	-1.33	-1.25
SLOPE	.01195	.01130	.01243	.01233	.01209	.01231	.01162	.01285	.01351	.01229
LOAD	-7.54	-8.96	-9.77	-8.37	-9.07	-9.62	-7.49	-8.46	-9.19	-8.72

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*                 HOUSTON PILE GROUP STUDY                  *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984        *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 5
CYCLE NO. 1001
LOAD ON GROUP = 155.94 FROM BIG LOAD CELL, KIPS
= 167.68 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = 2.29181 INCHES
WEST CAP DEFL = 2.26554 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	465.	395.	436.	522.	483.	469.	651.	481.	351.	434.
24.	702.	556.	642.	345.	584.	737.	819.	692.	529.	633.
36.	760.	715.	819.	1029.	754.	825.	1028.	879.	737.	839.
48.	921.	816.	953.	1173.	1119.	934.	1208.	1019.	826.	991.
60.	1019.	927.	1096.	1286.	1146.	1156.	1347.	1159.	829.	1132.
72.	1079.	1065.	1146.	1394.	1289.	1193.	1365.	1276.	995.	1200.
84.	1075.	1047.	1228.	1382.	1310.	1307.	1428.	1297.	1071.	1243.
96.	1159.	1136.	1253.	1407.	1358.	1236.	1363.	1315.	1087.	1247.
114.	1081.	1121.	1231.	1234.	1199.	1221.	1128.	1139.	1043.	1151.
132.	673.	1020.	1104.	910.	683.	951.	892.	923.	839.	944.
156.	463.	555.	579.	397.	567.	508.	390.	448.	493.	493.
AT LOAD POINT:										
DEFL	2.34	2.34	2.41	2.33	2.36	2.38	2.39	2.39	2.41	2.37
SLOPE	-.02413	-.02147	-.02336	-.02476	-.02345	-.02350	-.02515	-.02463	-.02410	-.02334
LOAD	17.86	13.32	16.89	25.88	17.65	18.53	24.84	17.23	15.44	18.63

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*                  HOUSTON PILE GROUP STUDY                *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984       *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 5
CYCLE NO. 1005
LOAD ON GROUP = 139.75 FROM BIG LOAD CELL, KIPS
= 150.46 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = 1.98336 INCHES
WEST CAP DEFL = 1.95004 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	411.	372.	386.	553.	423.	414.	583.	425.	311.	431.
24.	613.	581.	560.	736.	599.	66[illegible].	718.	610.	470.	615.
36.	670.	657.	714.	9[illegible].	685.	731.	961.	779.	627.	744.
48.	799.	779.	833.	1029.	895.	86[illegible].	1065.	909.	741.	831.
60.	694.	884.	950.	1134.	1119.	995.	1185.	1027.	741.	931.
72.	948.	101[illegible].	1016.	1229.	1151.	1074.	1198.	1134.	890.	1072.
84.	950.	1012.	1096.	1219.	1171.	1179.	1262.	1159.	960.	1112.
96.	982.	1044.	1109.	1245.	1115.	1129.	1159.	1177.	959.	1113.
114.	971.	1033.	1100.	1095.	1178.	1078.	1065.	1062.	920.	1038.
132.	778.	92[illegible].	989.	787.	871.	846.	789.	836.	731.	836.
156.	401.	445.	499.	346.	442.	419.	337.	375.	420.	414.
AT LOAD POINT:										
DEFL	2.04	2.10	2.1[illegible]	2.02	2.06	2.07	2.08	2.00	2.09	2.07
SLOPE	-.02127	-.02205	-.02055	-.02173	-.02054	-.02078	-.02213	-.02159	-.02092	-.02103
LOAD	15.77	14.83	14.9[illegible]	21.19	15.82	16.57	22.00	15.67	13.55	16.72

* HOUSTON PILE GROUP STUDY *
* FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *

	NORTH	
D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 5
CYCLE NO. 1010
LOAD ON GROUP = 147.59 FROM BIG LOAD CELL, KIPS
= 157.82 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = 2.17549 INCHES
WEST CAP DEFL = 2.12273 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	441.	429.	413.	583.	450.	437.	606.	450.	327.	460.
24.	657.	619.	631.	784.	539.	695.	761.	548.	495.	655.
36.	726.	733.	768.	958.	734.	773.	959.	828.	651.	793.
48.	863.	838.	899.	1099.	962.	932.	1137.	970.	783.	943.
60.	965.	949.	1044.	1214.	1092.	1056.	1274.	1135.	787.	1054.
72.	1024.	1079.	1091.	1318.	1229.	1141.	1291.	1217.	946.	1149.
84.	1023.	1048.	1172.	1309.	1252.	1252.	1358.	1242.	1020.	1191.
96.	1057.	1128.	1198.	1341.	1300.	1243.	1245.	1262.	1035.	1195.
114.	1039.	1104.	1161.	1185.	1153.	1169.	1085.	1143.	934.	1117.
132.	842.	956.	1064.	867.	945.	923.	860.	845.	802.	909.
156.	445.	547.	556.	382.	544.	485.	374.	427.	470.	470.
AT LOAD POINT:										
DEFL	2.23	2.30	2.32	2.21	2.25	2.27	2.27	2.28	2.28	2.25
SLOPE	-.02303	-.02155	-.02223	-.02357	-.02239	-.02245	-.02393	-.02356	-.02303	-.02283
LOAD	16.63	15.56	15.87	22.47	16.61	17.35	22.94	16.30	14.09	17.54

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* HOUSTON PILE GROUP STUDY *
* FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
****

	NORTH	
D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 5
CYCLE NO. 2001
LOAD ON GROUP = -158.22 FROM BIG LOAD CELL, KIPS
= -149.57 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -2.09813 INCHES
WEST CAP DEFL = -2.11334 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-341.	-446.	-494.	-365.	-422.	-384.	-335.	-423.	-483.	-411.
24.	-563.	-673.	-720.	-567.	-602.	-599.	-539.	-609.	-671.	-619.
36.	-657.	-820.	-919.	-704.	-763.	-773.	-679.	-779.	-839.	-773.
48.	-760.	-955.	-1116.	-842.	-937.	-946.	-797.	-896.	-940.	-911.
60.	-849.	-1055.	-1149.	-981.	-1012.	-1081.	-876.	-978.	-1071.	-1006.
72.	-931.	-1095.	-1227.	-1036.	-1132.	-1232.	-1024.	-1175.	-1190.	-1131.
84.	-994.	-1121.	-1250.	-1119.	-1136.	-1239.	-1115.	-1157.	-1177.	-1145.
96.	-1030.	-1092.	-1192.	-1103.	-1184.	-1262.	-1109.	-1170.	-1077.	-1135.
114.	-966.	-1014.	-1097.	-1069.	-1122.	-1038.	-1009.	-1065.	-880.	-1029.
132.	-856.	-773.	-830.	-895.	-887.	-847.	-869.	-816.	-653.	-825.
156.	-497.	-352.	-378.	-416.	-431.	-347.	-494.	-384.	-267.	-397.
AT LOAD POINT:										
DEFL	-2.13	-2.07	-2.03	-2.00	-2.06	-2.04	-2.08	-2.09	-2.12	-2.07
SLOPE	.02111	.02042	.02114	.01993	.02056	.02110	.02020	.02120	.02214	.02087
LOAD	-13.56	-16.63	-19.35	-15.76	[illegible]	-17.30	-15.99	-14.35	-19.33	-16.62

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*              HOUSTON PILE GROUP STUDY              *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 5
CYCLE NO. 2.05
LOAD ON GROUP = -137.96 FROM BIG LOAD CELL, KIPS
= -127.61 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -2.09751 INCHES
WEST CAP DEFL = -2.09756 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-301.	-392.	-426.	-323.	-380.	-340.	-300.	-374.	-425.	-352.
24.	-448.	-601.	-530.	-510.	-548.	-538.	-493.	-544.	-599.	-545.
36.	-599.	-738.	-819.	-639.	-706.	-735.	-628.	-706.	-767.	-731.
48.	-699.	-886.	-1019.	-771.	-877.	-875.	-737.	-925.	-870.	-840.
60.	-790.	-979.	-1073.	-939.	-951.	-1006.	-821.	-911.	-1005.	-939.
72.	-873.	-1030.	-1169.	-968.	-1144.	-1157.	-965.	-1014.	-1133.	-1039.
84.	-941.	-1072.	-1200.	-1059.	-1134.	-1173.	-1060.	-1103.	-1135.	-1092.
96.	-968.	-1065.	-1163.	-1053.	-1133.	-1207.	-1063.	-1124.	-1051.	-1094.
114.	-942.	-1000.	-1092.	-1038.	-1037.	-1009.	-980.	-1043.	-876.	-1017.
132.	-856.	-775.	-845.	-893.	-834.	-854.	-868.	-831.	-669.	-831.
156.	-527.	-381.	-409.	-454.	-520.	-38 .	-516.	-423.	-284.	-433.
AT LOAD POINT:										
DEFL	-2.12	-2.05	-2.03	-1.98	-2.05	-2.03	-2.03	-2.08	-2.11	-2.05
SLOPE	.02056	.01974	.02042	.01930	.02000	.02048	.01965	.02057	.02143	.02024
LOAD	-10.73	-13.85	-15.43	-13.88	-14.61	-15.22	-14.39	-12.23	-16.42	-14.20

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*             HOUSTON PILE GROUP STUDY                  *
* FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984     *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 5
CYCLE NO. 2?10
LOAD ON GROUP = -127.33 FROM BIG LOAD CELL, KIPS
= -117.27 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -2.07533 INCHES
WEST CAP DEFL = -2.07656 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-280.	-364.	-393.	-302.	-356.	-315.	-282.	-350.	-396.	-338.
24.	-418.	-561.	-585.	-480.	-520.	-503.	-467.	-510.	-563.	-512.
36.	-564.	-692.	-766.	-594.	-675.	-664.	-597.	-655.	-726.	-652.
48.	-660.	-837.	-963.	-751.	-641.	-823.	-702.	-782.	-830.	-797.
60.	-752.	-931.	-1022.	-868.	-914.	-9[illegible].	-785.	-867.	-969.	-895.
72.	-835.	-937.	-1122.	-929.	-1[illegible]7.	-111[illegible].	-929.	-970.	-1099.	-939.
84.	-908.	-1036.	-11[illegible].	-1023.	-1051.	-1134.	-1027.	-1063.	-1109.	-1057.
96.	-964.	-1035.	-1132.	-1025.	-1105.	-1177.	-1038.	-1090.	-1034.	-1057.
114.	-930.	-98[illegible].	-1082.	-1020.	-10[illegible]8.	-935.	-966.	-1025.	-872.	-994.
132.	-855.	-776.	-853.	-893.	-879.	-858.	-867.	-835.	-672.	-832.
156.	-537.	-38[illegible].	-424.	-464.	-528.	-394.	-523.	-441.	-290.	-445.

AT LOAD POINT:

	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
DEFL	-2.10	-2.13	-2.0[illegible]	-1.98	-2.02	-2.[illegible]	-2.04	-2.05	-2.08	-2.03
SLOPE	.02107	.01922	.01998	.01865	.01956	.01998	.01922	.[illegible]2004	.02094	.01975
LOAD	-9.70	-12.58	-14.93	-12.91	-13.57	-13.91	-13.44	-11.23	-13.01	-13.03

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*                 HOUSTON PILE GROUP STUDY                     *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984           *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 5
CYCLE NO. 2120
LOAD ON GROUP = -132.07 FROM BIG LOAD CELL, KIPS
= -121.65 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -2.10835 INCHES
WEST CAP DEFL = -2.11782 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-285.	-371.	-406.	-339.	-365.	-319.	-287.	-356.	-404.	-344.
24.	-424.	-571.	-592.	-489.	-527.	-506.	-473.	-516.	-570.	-519.
36.	-570.	-699.	-773.	-611.	-682.	-666.	-603.	-670.	-734.	-669.
48.	-665.	-842.	-966.	-739.	-849.	-828.	-738.	-785.	-837.	-812.
60.	-757.	-934.	-1024.	-875.	-922.	-955.	-791.	-867.	-975.	-900.
72.	-839.	-996.	-1120.	-934.	-1015.	-1103.	-935.	-968.	-1106.	-1001.
84.	-913.	-1039.	-1155.	-1030.	-1058.	-1127.	-1034.	-1062.	-1115.	-1060.
96.	-970.	-1040.	-1129.	-1037.	-1115.	-1172.	-1050.	-1091.	-1041.	-1072.
114.	-941.	-1032.	-1088.	-1030.	-1051.	-1033.	-982.	-1031.	-884.	-1015.
132.	-869.	-794.	-875.	-910.	-894.	-881.	-885.	-857.	-588.	-833.
156.	-551.	-404.	-446.	-523.	-544.	-434.	-537.	-463.	-332.	-457.
AT LOAD POINT:										
DEFL	-2.14	-2.07	-2.04	-2.10	-2.05	-2.04	-2.08	-2.11	-2.12	-2.07
SLOP	.02143	.01981	.02026	.01920	.01993	.02029	.01958	.02037	.02129	.02011
LOAD	-10.16	-13.23	-15.42	-13.45	-14.00	-14.34	-13.84	-11.64	-15.58	-13.52

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*                HOUSTON PILE GROUP STUDY                   *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984        *
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NORTH

J	G	A
F	L	H
B	I	C

PILE LEGEND

LOAD NO. 5
CYCLE NO. 2050
LOAD ON GROUP = -124.03 FROM BIG LOAD CELL, KIPS
= -114.19 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -2.05954 INCHES
WEST CAP DEFL = -2.10959 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-263.	-343.	-368.	-288.	-343.	-298.	-267.	-331.	-374.	-319.
24.	-392.	-530.	-547.	-457.	-498.	-475.	-445.	-481.	-530.	-434.
36.	-531.	-651.	-717.	-573.	-647.	-627.	-568.	-526.	-687.	-625.
48.	-620.	-791.	-903.	-592.	-807.	-781.	-667.	-733.	-739.	-754.
60.	-713.	-853.	-961.	-823.	-876.	-904.	-747.	-812.	-929.	-850.
72.	-796.	-942.	-1058.	-879.	-987.	-1049.	-890.	-911.	-1053.	-951.
84.	-876.	-1014.	-1097.	-974.	-1115.	-1076.	-995.	-1008.	-1032.	-1014.
96.	-941.	-1024.	-1079.	-936.	-1076.	-1127.	-1027.	-1051.	-1019.	-1037.
114.	-934.	[illegible]	-1064.	-994.	-1066.	-993.	-977.	-1023.	-886.	-993.
132.	-878.	-799.	-916.	-916.	-898.	-834.	-893.	-872.	-733.	-852.
156.	-571.	-413.	-460.	-558.	-561.	-453.	-551.	-489.	-317.	-439.
AT LOAD POINT:										
DEFL	-2.13	-2.16	-2.03	-1.99	-2.05	-2.03	-2.07	-2.08	-2.11	-2.05
SLOPE	.02002	.01922	.01978	.01877	.01953	.01934	.01920	.01988	.02082	.01957
LOAD	-9.60	-12.40	-14.29	-12.79	-13.27	-13.50	-13.07	-10.85	-14.43	-12.59

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*              HOUSTON PILE GROUP STUDY             *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
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NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 5
CYCLE NO. 2100
LOAD ON GROUP = -115.21 FROM BIG LOAD CELL, KIPS
= -107.63 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -2.09920 INCHES
WEST CAP DEFL = -2.10526 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-245.	-324.	-344.	-272.	-328.	-283.	-248.	-311.	-347.	[illegible]
24.	-365.	-532.	-513.	-435.	-477.	-453.	-416.	-452.	-493.	-457.
36.	-497.	-615.	-674.	-546.	-622.	-601.	-532.	-588.	-641.	-591.
48.	-581.	-753.	-655.	-661.	-776.	-75 .	-626.	-689.	-744.	-715.
60.	-677.	-847.	-916.	-789.	-844.	-871.	-703.	-751.	-835.	-810.
72.	-761.	-919.	-1018.	-845.	-937.	-1018.	-846.	-859.	-1021.	-912.
84.	-846.	-977.	-1064.	-942.	-986.	-1044.	-958.	-961.	-1047.	-931.
96.	-917.	-1006.	-1055.	-957.	-1051.	-1047.	-1002.	-1012.	-997.	-1011.
114.	-925.	-994.	-1051.	-978.	-1052.	-973.	-972.	-1014.	-887.	-934.
132.	-883.	-795.	-914.	-919.	-894.	-893.	-903.	-888.	-721.	-858.
156.	-587.	-421.	-497.	-570.	-571.	-475.	-566.	-516.	-335.	-504.
AT LOAD POINT:										
DEFL	-2.13	-2.14	-2.03	-1.97	-2.04	-2.01	-2.05	-2.08	-2.10	-2.05
SLOPE	.01967	.01383	.01935	.01849	.01921	.01949	.01885	.01948	.02039	.01931
LOAD	-9.04	-11.76	-13.28	-12.25	-11.64	-12.89	-12.31	-13.13	-13.32	-11.96

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*              HOUSTON PILE GROUP STUDY                     *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984        *
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NORTH

D	G	A
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B	I	C

PILE LEGEND

LOAD NO. 5
CYCLE NO. 2200
LOAD ON GROUP = -107.69 FROM BIG LOAD CELL, KIPS
= -102.36 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -2.10974 INCHES
WEST CAP DEFL = -2.10572 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-235.	-309.	-323.	-262.	-316.	-271.	-234.	-294.	-313.	-284.
24.	-351.	-479.	-482.	-421.	-461.	-435.	-399.	-427.	-445.	-433.
36.	-479.	-591.	-635.	-529.	-603.	-582.	-508.	-556.	-583.	-563.
48.	-562.	-724.	-812.	-641.	-754.	-729.	-599.	-652.	-633.	-634.
60.	-656.	-816.	-877.	-767.	-823.	-849.	-674.	-722.	-824.	-779.
72.	-739.	-882.	-983.	-823.	-916.	-994.	-816.	-817.	-961.	-831.
84.	-826.	-956.	-1036.	-922.	-966.	-1023.	-936.	-921.	-999.	-953.
96.	-897.	-1001.	-1034.	-943.	-1333.	-1077.	-981.	-977.	-955.	-939.
114.	-917.	-994.	-1045.	-967.	-1342.	-967.	-968.	-1093.	-892.	-977.
132.	-688.	-797.	-955.	-920.	-593.	-891.	-912.	-936.	-754.	-877.
156.	-606.	-431.	-522.	-573.	-582.	-486.	-583.	-550.	-401.	-525.
AT LOAD POINT:										
DEFL	-2.14	-2.04	-2.04	-1.97	-2.04	-2.11	-2.07	-2.09	-2.11	-2.15
SLOPE	.01957	.01889	.01918	.01838	.01955	.01926	.01848	.01928	.01997	.01911
LOAD	-8.81	-11.23	-12.41	-11.92	-12.23	-12.42	-11.62	-9.55	-11.98	-11.37

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*                 HOUSTON PILE GROUP STUDY                    *
*  FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984          *
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		NORTH	
	D	G	A
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	B	I	C
	PILE LEGEND		

LOAD NO. 6
CYCLE NO. 2.01
LOAD ON GROUP = -9.52 FROM BIG LOAD CELL, KIPS
= -5.34 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.35572 INCHES
WEST CAP DEFL = -.41586 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-12.	-35.	-35.	-6.	-27.	7.	3.	-31.	-21.	-19.
24.	-20.	-55.	-48.	-32.	-39.	-3.	-32.	-38.	-41.	-39.
36.	-43.	-63.	-61.	-58.	-59.	-36.	-31.	-51.	-43.	-51.
48.	-38.	-93.	-100.	-59.	-91.	-60.	-43.	-53.	-40.	-54.
60.	-55.	-58.	-98.	-89.	-51.	-75.	-28.	-33.	-77.	-56.
72.	-50.	-77.	-132.	-90.	-62.	-120.	-57.	-41.	-89.	-81.
84.	-84.	-92.	-130.	-128.	-65.	-98.	-73.	-77.	-90.	-93.
96.	-106.	-121.	-107.	-83.	-83.	-155.	-76.	-75.	-51.	-95.
114.	-110.	-96.	-95.	-51.	-94.	-99.	-90.	-82.	-117.	-94.
132.	-133.	-53.	-89.	-91.	-108.	-136.	-94.	-104.	-121.	-107.
156.	-153.	-71.	-80.	-46.	-110.	-91.	-111.	-117.	-84.	-100.
AT LOAD POINT:										
DEFL	-.34	-.27	-.23	-.25	-.29	-.25	-.27	-.24	-.28	-.27
SLOPE	.00315	.00239	.00246	.00256	.00228	.00244	.00249	.00229	.00237	.00251
LOAD	-.36	-.78	-.69	-.41	-.89	-.74	-.62	-.36	-.60	-.59

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* HOUSTON PILE GROUP STUDY *
* FIELD DATA FROM LATERAL LOAD TEST OF MAY 17, 1984 *
****

NORTH

D	G	A
F	E	H
B	I	C

PILE LEGEND

LOAD NO. 6
CYCLE NO. 2005
LOAD ON GROUP = -29.16 FROM BIG LOAD CELL, KIPS
= -24.93 FROM SUM OF PILE LOAD CELLS
EAST CAP DEFL = -.64474 INCHES
WEST CAP DEFL = -.87970 INCHES

BENDING MOMENTS, INCH-KIPS

DEPTH, INCHES	PILE A	PILE B	PILE C	PILE D	PILE E	PILE F	PILE G	PILE H	PILE I	AVG
12.	-57.	-94.	-87.	-60.	-82.	-43.	-49.	-83.	-77.	-70.
24.	-90.	-153.	-127.	-130.	-124.	-85.	-115.	-116.	-120.	-118.
36.	-137.	-177.	-165.	-158.	-181.	-147.	-144.	-155.	-146.	-157.
48.	-152.	-234.	-232.	-187.	-233.	-195.	-174.	-181.	-171.	-195.
60.	-193.	-251.	-255.	-243.	-220.	-237.	-180.	-187.	-235.	-223.
72.	-211.	-252.	-313.	-265.	-262.	-315.	-245.	-225.	-275.	-256.
84.	-266.	-323.	-336.	-330.	-254.	-323.	-290.	-287.	-336.	-315.
96.	-309.	-367.	-337.	-309.	-327.	-380.	-322.	-311.	-297.	-330.
114.	-338.	-351.	-363.	-322.	-352.	-345.	-340.	-345.	-349.	-345.
132.	-372.	-273.	-357.	-334.	-343.	-366.	-335.	-349.	-322.	-339.
156.	-291.	-159.	-230.	-223.	-256.	-214.	-244.	-251.	-185.	-229.
AT LOAD POINT:										
DEFL	-.84	-.75	-.73	-.72	-.77	-.72	-.75	-.79	-.78	-.75
SLOPE	.00763	.00672	.00689	.00692	.00670	.00717	.00691	.00685	.00681	.00694
LOAD	-2.06	-3.16	-2.82	-2.92	-3.14	-2.92	-3.26	-1.9[illegible]	-2.74	-2.77

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